5.0 Hydraulic Modeling

Paragraph 12 of the CD defines the requirements of the collection and transmission system model. Consent Decree paragraph 12E requires a certification that the sewershed model includes the elements required in paragraphs 12A and B. The model is capable of and can be used for predicting the volume of wastewater flow, the hydraulic grade line (water levels) at any point in the modeled system, the capacity of the system, and the locations where overflows are likely. The model configuration is based on representative, accurate, and verified system attribute data. The model has been calibrated and validated with spatially and temporally representative rainfall and flow data collected during the flow monitoring program.

Modeling requirements per the Consent Decree are defined in even greater detail in the BaSES Manual, Section 7 (Hydraulic Modeling). The appropriate sections of the BaSES manual will be cited throughout this report to clearly identify how the development, calibration, and application of the Outfall Sewershed model fulfills the requirement of the BaSES manual and the objectives of the Consent Decree.

Following the guidance of Paragraph 12.B of the CD, the Outfall Sewershed model is capable of predicting:

- The volume of wastewater flow in the major gravity lines,
- Hydraulic pressure or hydraulic grade line of wastewater at any point in the major gravity lines,
- Likelihood and location of overflows under high flow conditions and considering normal in-line storage capacity.

The model is also:

- Configured based on representative, accurate, and verified system attribute data (i.e., pipe sizes and invert elevations, manhole rim elevations, etc.),
- Calibrated using spatially and temporally representative rainfall data and flow data obtained during the rainfall and flow monitoring, and
- Verified using spatially and temporally representative rainfall data and flow data; that data shall be independent of the data used to calibrate the model.

5.1 Model Network

In general, a hydraulic model contains three essential components:

- Network of sewer infrastructure (pipes, pumps and structures);
- Tributary basins served by the sewer network (i.e., the source of flows to the network), and

- Boundary conditions (i.e., upstream inflows and downstream water levels that represent the system beyond the model boundaries).

Figure 5.1 is a schematic of the large diameter trunk sewers in the Outfall Sewershed. The schematic shows the points of inflow to the Outfall Sewershed model from the upstream sewersheds and the locations of the downstream boundary conditions at the County Line.

Approximate capacities of the trunk sewers are noted on the schematic for a clean condition if sediment were removed and for the existing condition with sediment. The capacities are given as ranges to account for the variable depth of sedimentation in the existing condition and for a possible range of pipe roughness values in the clean condition (Manning's roughness from 0.015 to 0.013). The representative inflow rates noted on the schematic are approximate values for the typical inflows from upstream sewersheds in a large wet weather event; these values are for conceptual reference. Inflow hydrographs provided by the technical program manager were used for the model simulations.

The Joint Venture Team used the InfoWorks™ CS hydraulic modeling software by Wallingford Software to build a hydraulic network model of the Outfall Sewershed. The InfoWorks™ model satisfies the requirements of Consent Decree paragraph 12B and is useful to perform a dynamic hydraulic evaluation of the sewer system in accordance with Paragraph 9.F of the Consent Decree.

Consent decree paragraph 9.F(i) gives general instructions for the model development, which are specified in greater detail in BaSES 7.4.1. The model network contains:

- All gravity lines that are 10-inches in diameter or larger
- All 8-inch sewer lines that convey or are necessary to accurately represent flow attributable to a service area in each of the collection system sewershed service areas
- All gravity sewer lines that convey wastewater from one pumping station service area to another pumping station service area
- All gravity sewer lines that have caused or contributed to, or that the City knows are likely to cause or contribute to capacity-related overflows
- All manholes, junctions, and structures along modeled sewer lines
- Simulated control structures (gates, weirs, pump stations) as they exist in the field

In general the network extent is adequately defined by the 10-inch pipes; therefore, only a few 8-inch pipes are included in the model. There are no pump stations or other control structures in the Outfall Sewershed.

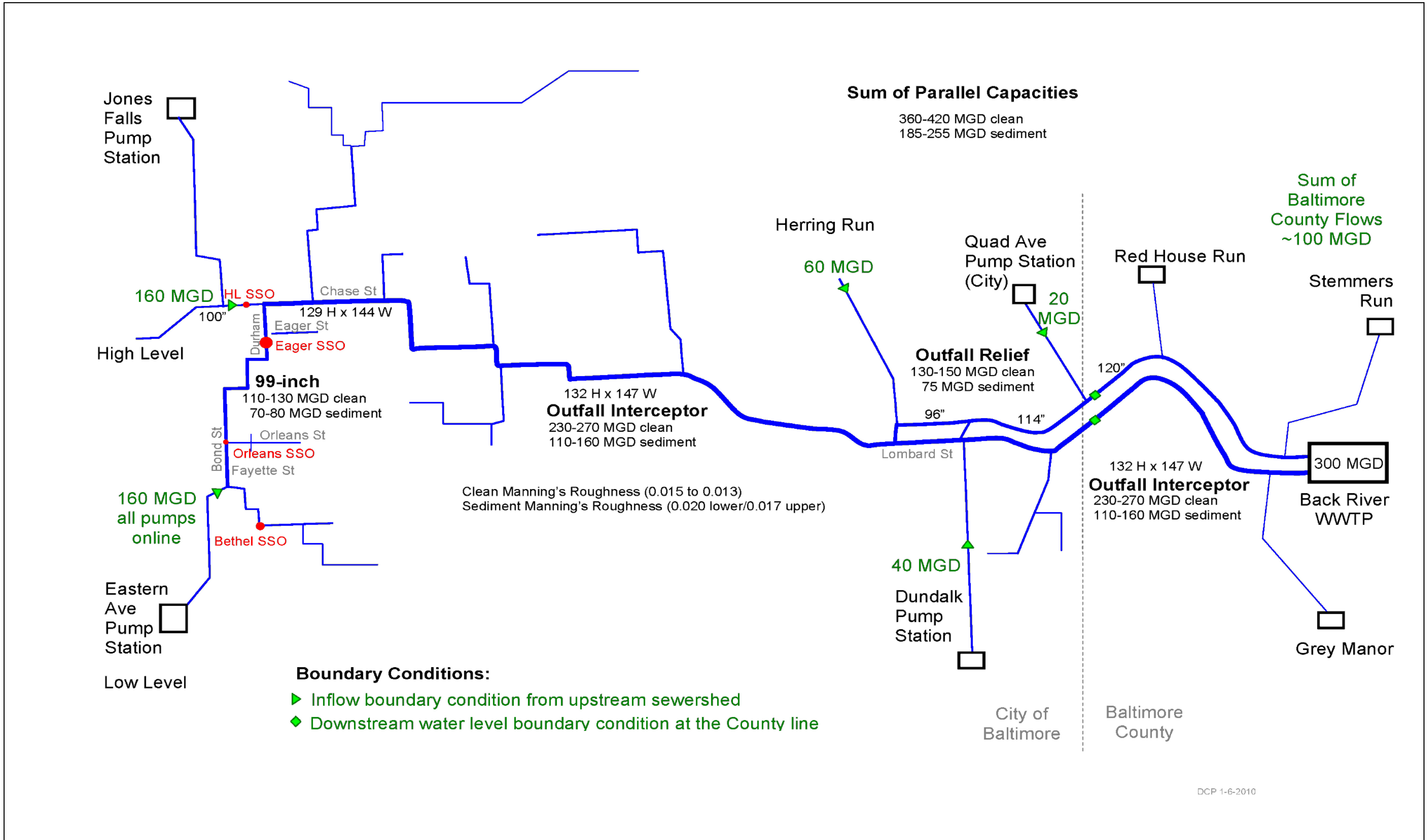


Figure 5.1 Schematic of Outfall Sewershed Trunk Sewers

The configuration of the model is based on GIS data that is developed from field surveyed data (supplemented by as-built drawings). This is to satisfy the Consent Decree paragraph 12.B-(ii)-(a) requirement that the system configuration be based on system attribute data that is representative, accurate and verified.

In accordance with BaSES 7.4.2, the Maryland State Plane Coordinate System (NAD83-Feet) is used for the horizontal datum. The vertical datum for the model is the North American Vertical Datum of 1988 (NAVD88).

Sewer service areas (SSAs) were initially defined by the City, but later refined into model subcatchments to meet the requirements of the CD. The subcatchments are essentially the same as the sewer service areas (SSAs); however, some of the SSAs have been further subdivided to accommodate the need to load flow to each branch of the network. SSAs were also divided due to the locations of the flow meters. These changes follow the guidelines from Section 7.4.4 of the BaSES Manual.

For calibration, the subcatchments are grouped according to the flow meter to which they are tributary. In general, all subcatchments within a flow meter basin have the same calibration parameters.

Sanitary flow and infiltration and inflow (I/I) contribute to the total flow conveyed by the collection system. The model defines the sanitary flow (with a diurnal pattern) and the I/I response to wet weather using parameters in the model subcatchments.

Dry weather flow is discussed in the BaSES Manual Sections 3.2.3, 3.5 and 7.4.5. The objective of the dry weather flow development is to characterize the dry weather flow pattern so that during wet weather conditions it is possible to distinguish between flow due to infiltration and inflow (I/I) and the base sanitary flow. Calibration objectives focus on properly simulating the volume, diurnal peaks, and the timing of the diurnal pattern during dry weather conditions. Water consumption patterns and groundwater infiltration vary over time; much of the variability is periodic and repeated. The model development aims to represent the typical quantity and variability of dry weather flow by a fixed set of parameters. The model cannot duplicate all of the flow patterns or periods of irregular flow; instead it is an approximate match to the dominant dry weather flow characteristics.

Base sanitary flows (BSF) have been developed by the City's Technical Program Management Team for each SSA (as described in the BaSES manual section 7.4.5). The BSF values represent the sanitary flow generated by users. The average dry day flow (ADF) from each SSA is the sum of the BSF and any groundwater infiltration (GWI).

ADF = BSF + GWI

The ADF is estimated from the flow meter data stored in Sliicer. The GWI is estimated as a calibration parameter to achieve a good match between the simulated flows and the measured flows during dry weather. The estimated GWI determined for each meter basin is assigned to the tributary subcatchments in proportion to area.

BaSES manual section 7.4.6 defines the modeling approach used to simulate the wet weather flow. This modeling approach assumes a direct relationship between rainfall and the wet weather flow response in the sewer system. The details of this deterministic relationship are described below; however, it is important to note that the modeling approach does not account for variable antecedent soil moisture conditions. The model calibration assumes that the hydrologic conditions experienced during the monitoring period are representative of typical hydrologic conditions. Special hydrologic conditions may not be properly modeled using this methodology (such as, events with significant snow melt, back to back events with a prolonged series of significant storms, or extreme events such as hurricane related storms).

The development of the model is based on rainfall and flow meter data. Uncertainties in both the rainfall and flow meter measurements are compounded in the process of developing a model relationship between the two. The uncertainties are not just due to the accuracies of the instruments, but also to the intrinsic variability of the quantities being measured. For example, rainfall measured at a gauge may or may not be a sufficient representation of the rainfall over the meter basin to which it is assigned. The rainfall and flow are measured at spatially separate locations. Overall, the correlation can be derived from the data by calibrating the model to many events. The objective of the calibration is to choose model parameters that realistically characterize the basin response to rainfall for the most probable conditions, even though the match may not be ideal for each and every event in the measurement record. The use of radar rainfall estimates seeks to improve the correlation between rainfall patterns and flow meter response, but rainfall is just one of many sources of variability.

Rainfall derived infiltration and inflow (RDII) is simulated using the SWMM RUNOFF routines in InfoWorks™. The following parameters are needed for each subcatchment in the model to develop wet-weather flows:

- Area
- R-Value
- Depression Storage
- Width
- Slope
- Overland Flow Routing Coefficients

Area

The **Contributing Area** parameter represents the area of each subcatchment, in acres, that is served by the collection system. Areas that are not sewered (i.e. cemeteries, golf courses, parks, etc.) are deducted from the total area of subcatchments to determine the contributing area.

R-Value

The R-Value represents the fraction of the rainfall that enters the sewer system. Sliicer provides an initial estimate of the R-Value for each flow meter basin by plotting the RDII volume versus the rainfall depth (Q vs. I plot) and then developing the best-fit linear regression line (the R-Value is based on the slope of the regression line). In the InfoWorks™ model, the R-Value is input as the **Fixed Runoff Coefficient**. Once in the model, this coefficient may be adjusted to refine the calibration based on the routed simulated response in the model. This provides a more accurate prediction of flow volume.

The equation for I/I volume using the R-value is:

$V = K\ R\ A\ (D-DS)$
Where V = Volume of I/I
K = a unit conversion constant = 1 MG/36.8 acre inches
R = dimensionless ratio of RDII volume to rainfall volume
A = contributing metershed area (acres)
D = rainfall depth (inches)
DS = depression storage (otherwise known as initial rainfall abstraction) (inches)

Depression Storage

Depression storage represents the amount of rainfall (inches) that is lost to surface wetting, ponding, interception, and evaporation during a storm; this parameter is also commonly known as the "initial abstraction". Depression storage is estimated by the location where the linear regression line intercepts the x-axis of the Sliicer software's Q versus I Plot. Typical values range from 0 to 0.5 inches, but can vary greatly for the same area depending on the antecedent moisture conditions. The depression storage value is entered into the appropriate Runoff Surface under the **Initial Loss Value** field of the InfoWorks™ model.

Width

The subcatchment width, known as the **Dimension** value in InfoWorks™, is a key calibration parameter that does not have a direct correlation to the actual dimensions of the subcatchment. During calibration, the subcatchment width value is adjusted so that the magnitude and time-to-peak of the simulated flow matches the observed peak flow in the monitoring data (peak RDII flow) for several storm events. Subcatchment width can greatly alter the shape of the hydrograph without impacting the volume. Because the width is directly proportional to the peak flow rate, its value may be adjusted as necessary to match the observed peak flows.

Slope

The **Slope** parameter is given a nominal value similar to the physical slope of the ground surface, but when the SWMM model is being used to simulate RDII, this parameter is no longer physically-based. Slope is a not a sensitive calibration parameter.

Overland Flow Routing Coefficient

The **Overland Flow Routing Coefficient,** also known as the Manning's Roughness Coefficient (n), is a secondary parameter that can be used to alter the shape of the hydrograph. A nominal value of 0.013 was used in the model for all subcatchments; however, this is not a sensitive parameter.

5.2 Model Calibration

BaSES manual Section 7.5 defines the objectives and criteria to be used for the calibration of the dry and wet weather flows. The calibration compares the simulated flows and water levels in the InfoWorks™ model to the measured flows and levels at the monitoring sites. A schematic of the meterbasins, previously given in Section 3, Figure 3.2.1, shows the relationship between the flow meters.

Subcatchments along the branch sewers in the Outfall Sewershed are calibrated using the meters located on the branch sewers. The remaining SSAs tributary to the major trunk sewers used nominal parameters to generate dry and wet weather flows. Meters located on the major trunk sewers are used to calibrate the large scale hydraulic properties and responses of the model (such as roughness, sediment, boundary conditions, and water depth). Thus there are two distinct applications of flow meter data to the model calibration; the smaller branch meters are used to calibrate the SSA flow generation parameters and the larger trunk meters are used to calibrate the large scale hydraulic parameters.

Attachment 5.2.1 is the Model Development and Calibration Report (MDCR) which contains complete details of the model development and the calibration performance.

Dry Weather Calibration

The dry weather calibration criteria are from BaSES manual Section 7.5. For a representative dry weather period, the simulated volume of flow should be within -10% to +20% of the measured volume and the peak dry weather flow rate should be within -10% to +20% of the measured flow rate. The timing of the peaks of the diurnal pattern should be within 1 hour of the measured peaks. Subjectively, the general shape of the diurnal pattern should be representative for most of the dry weather conditions.

The branch sewer meters were used to calibrate the SSAs; for these meters the dry weather comparison of the simulated results to the measured values is given in Table

5.2.1. Five dry weather periods (representing a sum of 114 days of dry weather flow) were used to develop the dry weather calibration parameters. The values presented in Table 5.2.1 summarize the results for a 7-day validation period from 12/4/2006 to 12/10/2006 (except for two meter sites that use other periods as is explained further below). In most cases the results satisfy the calibration criteria given in the BaSES manual; exceptions are explained in the MDCR.

Table 5.2.1 - Dry Weather Calibration Branch Sewer Meters Used to Calibrate Sewer Service Areas							
	Peak Flow				Volume (7 day duration)		
Meter	Measured (MGD)	Simulated (MGD)	Difference (MGD)	Percent Difference	Measured (MG)	Simulated (MG)	Percent Difference
HL01	0.43	0.36	-0.07	-16%	1.86	1.94	5%
HL02	0.67	0.64	-0.03	-4%	3.56	3.78	6%
HL03	0.80	0.89	0.08	10%	4.42	5.24	19%
HL04	0.64	0.60	-0.04	-6%	3.32	3.40	2%
HL05	0.40	0.39	-0.01	-2%	6.78	6.81	0%
OUT01	0.53	0.53	0.00	-3%	5.77	6.04	5%
OUT05	No data	0.19			No data	1.18	
OUT07	0.33	0.38	0.04	13%	1.29	1.82	41%
OUT08	0.62	0.60	-0.02	-4%	3.26	3.36	3%
OUT09	0.55	0.39	-0.16	-28%	1.87	1.82	-2%

Meter HL01 has a unique flow pattern with a strong weekly cycle that does not conform simply to a typical weekday/weekend pattern. Because of this, it is difficult to represent this pattern in the InfoWorks™ model. The selected calibration is a reasonable compromise to adapt the model diurnal flow pattern to the measured flow pattern.

Meters HL02, HL03, HL04 and HL05 are calibrated within the criteria for dry weather flow. Limited data was available for OUT01 beginning in February 2007; the results in the Table 5.1 are based on a dry weather period from 4/25/2007 to 5/10/2007.

No valid flow meter data is available for OUT05; the SSAs tributary to this meter basin have been assigned a flow that is two times the base sanitary flow (BSF) values provided by the City.

Meters OUT07 and OUT09 monitor the same area; OUT09 is a FlowShark meter located a few blocks downstream of OUT07 which is an Isco meter. The peak flow rate and volume at OUT07 and OUT09 should in principle be the same. The flow at these meter sites can be influenced by high water levels in the 99-inch Sewer that is downstream of this branch. The flow and water level in the 99-inch Sewer are largely controlled by the operations of the Eastern Avenue Pump Station. The flow meter data at OUT07 and OUT09 show the influence of the operations of the pump station. In general, it appears that OUT07 yields a better estimate of the peak flow rate while OUT09 yields a more consistent estimate of the volume of flow. The calibration results do not conform to the calibration criteria because of the uncertainty in the measured data.

Table 5.2.2 contains the dry weather calibration results for meters located on the major trunk sewers. The volumes in the table are for the 7-day period 12/4/2006 to 12/10/2006. These meters were used to calibrate the overall hydraulic response of the sewer network. The simulation results at meters along the major trunk sewers are highly sensitive to the assumed boundary condition values. Gaps or irregularities in the measured data used for the boundary condition propagate through the model.

The primary conclusion from this comparison is that the model is properly routing the input flow boundary conditions from the upstream sewersheds (that is, the measured flows from High Level, Jones Falls, Low Level, Herring Run, and Dundalk). The secondary benefit of this comparison is observations about the hydraulic consistency of the measured flow data. Most of the meters used in the large trunk sewers are FlowShark area-velocity meters; three of the meters are Isco area-velocity. In general, the FlowShark meters are better able to monitor the velocity in large pipes than the Isco meters (which are well suited to monitor flow in smaller pipes). Specific observations are noted below (progressing from the upstream to the downstream end).

Table 5.2.2 - Dry Weather Calibration

Major Trunk Sewer Meters Used to Evaluate Overall System Hydraulics (meters ordered from upstream to downstream)

	Peak Flow				Volume (7 day duration)		
Meter	Measured (MGD)	Simulated (MGD)	Difference (MGD)	Percent Difference	Measured (MG)	Simulated (MG)	Percent Difference
OUT06A[1]	32.41	45.80	13.39	41%	152.13	180.86	19%
OUT06[2]	45.73	44.08	-1.65	-4%	179.82	184.40	3%
TSHL01[2]	92.28	90.85	-1.43	-2%	517.98	521.66	1%
OUT04A[1]	64.87	91.59	26.71	41%	323.49	526.90	63%
OUT04[1]	82.69	91.26	8.57	10%	374.19	528.81	41%
OUT03[2]	90.69	91.41	0.72	1%	516.26	540.68	5%
OUT02[2]	91.40	90.38	-1.02	-1%	519.53	544.68	5%
TSOUT02[2]	100.00	76.40	-23.60	-24%	541.13	465.98	-14%
TSOUT01A[2]	99.20	76.87	-22.33	-23%	582.08	466.22	-20%
TSOUT01B[2]	44.66	40.36	-4.30	-10%	265.30	233.82	-12%

[1]Isco flow meter

[2]FlowShark meter

Meters OUT06A and OUT06 are located on the 99-inch Sewer that conveys flow from the Eastern Avenue Pump Station to the Outfall Sewer. OUT06A is located near the upstream end of the 99-inch Sewer close to the connection of the force main from the pump station. OUT06 is located near the downstream end of the 99-inch Sewer before connecting to the Outfall Sewer. The average dry weather flow in the 99-inch Sewer is approximately 26 MGD, but the flow is highly variable because the flow pattern is dominated by the pump station operations (typically varying from 10 to 40 MGD). The incremental flow from SSAs in the Outfall Sewershed between OUT06A and OUT06 is relatively small (only 0.2 MGD); therefore, the total flow at the two meters is essentially the same.

Meter TSHL01 is a FlowShark meter located near the upstream end of the Outfall Interceptor after the confluence of flows from the High Level/Jones Falls sewersheds and the 99-inch sewer from the Low Level sewershed. The average dry weather flow is approximately 74 MGD. As this data was used to develop the input boundary condition flows, the simulated and measured data agree very closely at TSHL01.

Meters OUT04A and OUT04 are Isco meters located on the Outfall Interceptor. The measured velocities (and consequently the recorded flow rate values) are consistently lower than values at neighboring meters (TSHL01 upstream and OUT03 downstream, both of which are FlowShark meters). It is the opinion of the hydraulic modeling engineers that the flow data at OUT04A and OUT04 have a low bias. It is assumed that the depth data is reasonable and that the flow pattern is realistic, but that measured flow values are lower than actual flows.

Meters OUT03 and OUT02 are FlowShark meters located along the Outfall Interceptor along Monument Street and Lombard Street, respectively. The simulated flows match the measured flows very well at both meter sites. OUT02 is located just upstream of the chamber that allows flow to divide between the 132-inch Outfall Interceptor and the 114-inch Relief Sewer. The average dry weather flow at OUT02 is approximately 78 MGD.

Flow from the Herring Run sewershed enters the model at the upstream end of the Outfall Relief sewer; the average dry weather flow at meter HR01 is approximately 18 MGD. Flow from the Dundalk sewershed enters the Outfall Interceptor just downstream of an inter-connection structure between the Outfall Interceptor and Outfall Relief Sewers. The average dry weather flow at meter TSDU03 is approximately 4 MGD. Based on this information, the sum of the flows from Herring Run, Dundalk and the Outfall Sewershed is approximately 100 MGD; this flow is conveyed by the parallel pipes (Outfall Interceptor and Outfall Relief Sewer) to the Baltimore County Line which is the downstream end of the Outfall Sewershed model.

Meters TSOUT01A and TSOUT01B are FlowShark meters located on the Outfall Interceptor and Outfall Relief Sewer, respectively, near the Baltimore County Line. The balance of flow between the two pipes is very sensitive to the water level boundary condition defined at the Baltimore County Line (the downstream nodes of model). In general the level, velocity, and flow data recorded for TSOUT01A and TSOUT01B are reasonable. The flow at both meter sites is regulated by the water level at the Back River WWTP. The flow at the County Line is subject to backwater conditions from the plant; the depth and velocity relationship does not follow the normal Manning's relationship for open channel flow (the depths are deeper and the velocities are slower than normal flow). This backwater influence is also apparent in the data for all of the major trunk sewer meters.

In addition to the balance of flow between the Outfall Interceptor and Outfall Relief sewer, there is some uncertainty in the magnitude of the measured flows. The measured average dry weather flows are 83 MGD at TSOUT01A and 38 MGD at TSOUT01B; the sum of the flows is 121 MGD. For comparison, the simulated average dry weather flows are 67 MGD at TSOUT01A (19% less than measured) and 33 MGD at TSOUT01B (13% less than measured); the sum of the simulated flows is 100 MGD (17% less than measured). Further efforts to refine the model calibration to increase the simulated flow at the County Line were not pursued because it would make the model less consistent with the other trunk sewer meters along the Outfall Interceptor. It is likely that there is a high bias in the measured flow values at TSOUT01A. This suspicion is supported by the data at meter TSOUT02.

Meter TSOUT02 is FlowShark meter located on the Outfall Interceptor just downstream of the connection from the Dundalk Sewershed. The flow at TSOUT02 and TSOUT01A are, in principle, equal flows. The measured average dry weather flow at TSOUT02 is 77 MGD, which is 6 MGD less than the measured flow at TSOUT01A. This further supports the assumption that meter TSOUT01A has a high bias in the measured flow values.

The simulated flows are consistent with the sum of the measured flows entering the parallel pipes from Herring Run, Dundalk, and the Outfall Sewershed (which is 97 MGD). Therefore, the calibration was defined to agree with as many meter sites as possible; in this case, however, the differences can be seen most clearly in the percent difference between simulated and measured values at TSOUT01A and TSOUT1B. In reality, the uncertainty could be (and likely is) shared between the various meters in the vicinity of the parallel sewers. The model configuration is, in the judgment of the Joint Venture engineers, a realistic representation of the flows and boundary conditions present in the system. The places where the difference between the simulated and measured values exceeds the calibration criteria are acceptable. This discussion of the dry weather flow response also assists with a proper interpretation of the wet weather calibration results.

Wet Weather Calibration

The wet weather calibration seeks to determine parameters that characterize the response in the sewer systems to wet weather conditions that cause I/I. During the 12-month calibration period (May 2006 to May 2007) there were 29 wet weather events identified as global storms. The radar rainfall data (CALAMAR) was used (when available) to drive the model simulations. Ground based rain gauge data was also used to run the simulations; this result provides a check on the radar rainfall simulation.

In general there are two types of significant wet weather events: (1) those that are driven by high intensity rainfall of a relatively short duration and (2) those that are driven by low intensity, longer duration rainfall. Because the modeling system chosen for this effort does not account for the influence of variable soil moisture storage, the simulated flows can either be calibrated to better match the short/high intensity storms or the longer/low intensity storm. The limitations of the modeling approach can not account for a wide variety of hydrologic conditions. Preference is given in this calibration to short/high intensity storms which drive the highest peak flows. This is also the type of event that was used to evaluate system capacity, as described later in this section.

The wet weather calibration criteria from the BaSES manual Section 7.5.2 are summarized in Table 5.2.3. In addition to the flow related comparisons (peaks and volumes), there are also criteria to evaluate the depth of flow. For pipes that are not surcharged, the simulated depth of flow should be within 4 inches of the measured depth of flow. For surcharged pipes the criteria depends on the size of pipe and whether the simulated flows are greater than or less than the measured depths.

Table 5.2.3: Wet Weather Validation Criteria

Simulated response	Percent difference from observed measurements
Peak Flow Rate	Within –10% and + 25%
Volume of Flow (assume duration from the start of rainfall to 2 days after rainfall ends)	Within –10% and + 20%
Depth of Flow in Surcharged Pipes: For pipes 21-inch diameter and larger For pipes smaller than 21-inch diameter	 Within -4 inches and +18 inches Within -4 inches and +6 inches
Depth of Flow in Unsurcharged Pipes	Within 4 inches
Shape and timing of hydrographs	Should be similar

The calibration results for each flow meter location are summarized in the MDCR with a time series plot and three statistical plots that compare the simulated results to the measured values. The statistical plots are a concise summary of the results that show the correlation between simulated results and observed values. Using meter OUT08 as an example, Figures 5.2.1, 5.2.2, and 5.2.3 are statistical plots for peak depth, peak flow, and volume for the wet weather events. Each statistical plot has a one-to-one line that represents perfect correlation between simulated and observed values. Upper and lower reference lines on the statistical plots show the envelope of the calibration criteria. When the pipe is not surcharged, the calibration criterion for peak depth is ±4 inches. When the pipe is surcharged, the calibration criterion is +18 inches and -4 inches because the pipe size is 24-inches. If the pipe diameter were less than 21 inches, the surcharged criteria would be +6 inches and -4 inches. Reference lines also mark the pipe crown to show surcharging when peak depth are greater than the pipe diameter.

For the larger surcharged events, the simulation results are within the calibration boundaries. When the pipe is not surcharged, the simulated peak depths are generally greater than observed depths. For a few of the smaller events in the transition zone, the model tends to simulate surcharging conditions for some events that did not have observed surcharging.

Each statistical plot shows the data points and two regression lines that have been fitted to the data points. One of the regression lines assumes a y-intercept of zero and the other allows for a y-intercept offset value. The equation and the goodness of fit correlation coefficient, R^2, are printed on the graph for each regression line. The correlation coefficient, R^2, is an indication of how well the model fits for a variety of wet weather conditions.

Figure 5.2.1 shows the simulated peak hydrograph depth compared to the measured flow depth. The simulated depths are typically higher during low flow periods, but are within the calibration criteria for the larger events with surcharging.

In Figure 5.2.2 the slope of the dotted red line for peak flow is 0.99, which means that the simulated peak flows are very close to the observed values overall. Reference lines on the plots of peak flow mark the calibration criteria of +25% and –10%.

In Figure 5.2.3 the slope of the dotted red line for the event volume is 1.05, which means that the simulations over predict the event volume by 5% on average. Reference lines on the statistical plots of event volume mark the calibration criteria of +20% and -10%.

Overall, the calibration of SSAs in meter basin OUT08 produce simulated results that are a realistic representation of the actual system hydraulics. The calibration results at OUT08 are representative of the overall calibration of the model at the other meter sites as well.

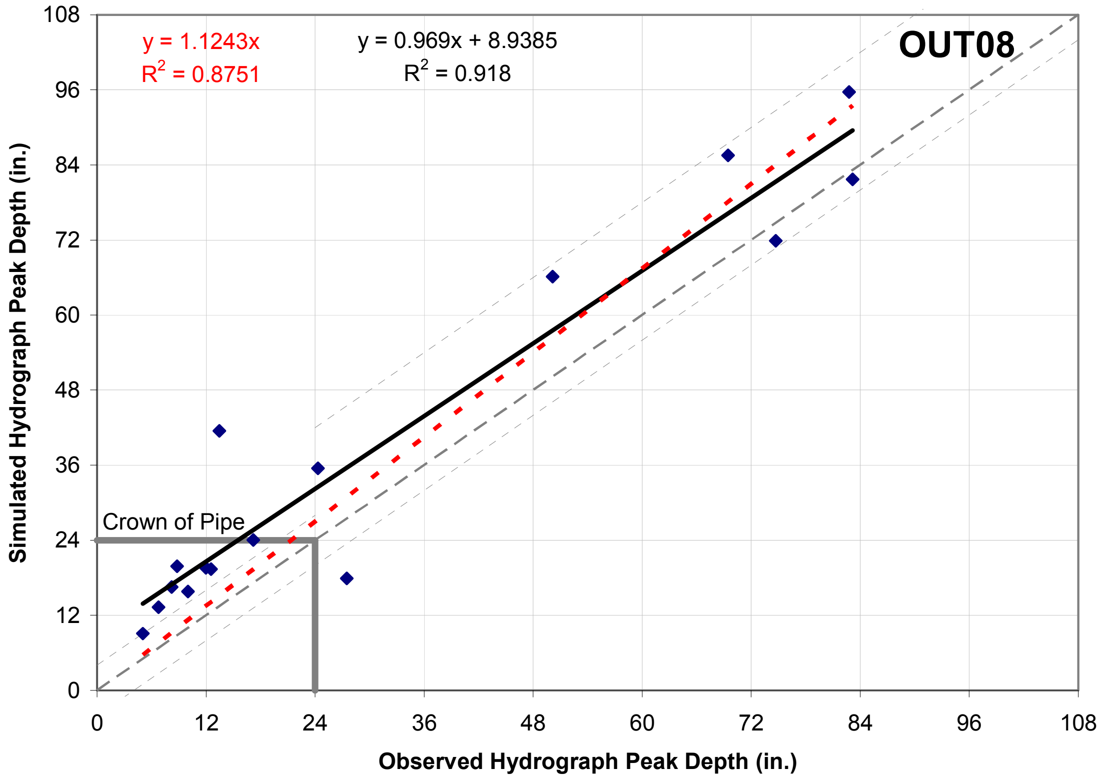


Figure 5.2.1. Statistical Plot of Peak Depth for OUT08.

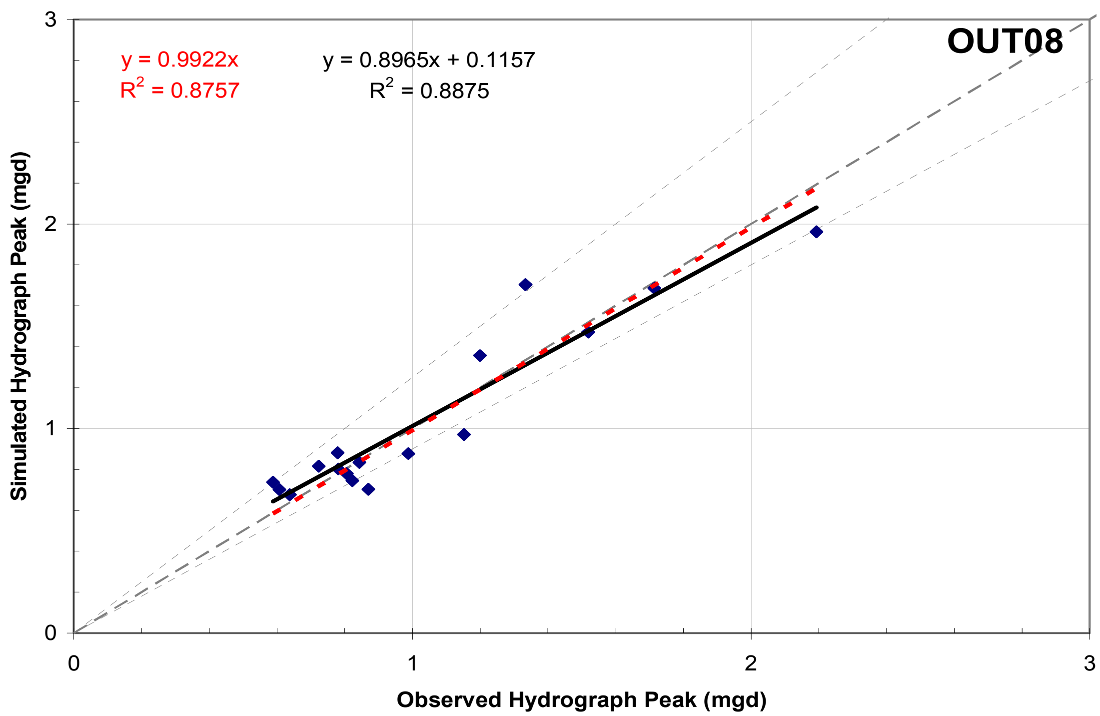


Figure 5.2.2. Statistical Plot of Peak Flow for OUT08.

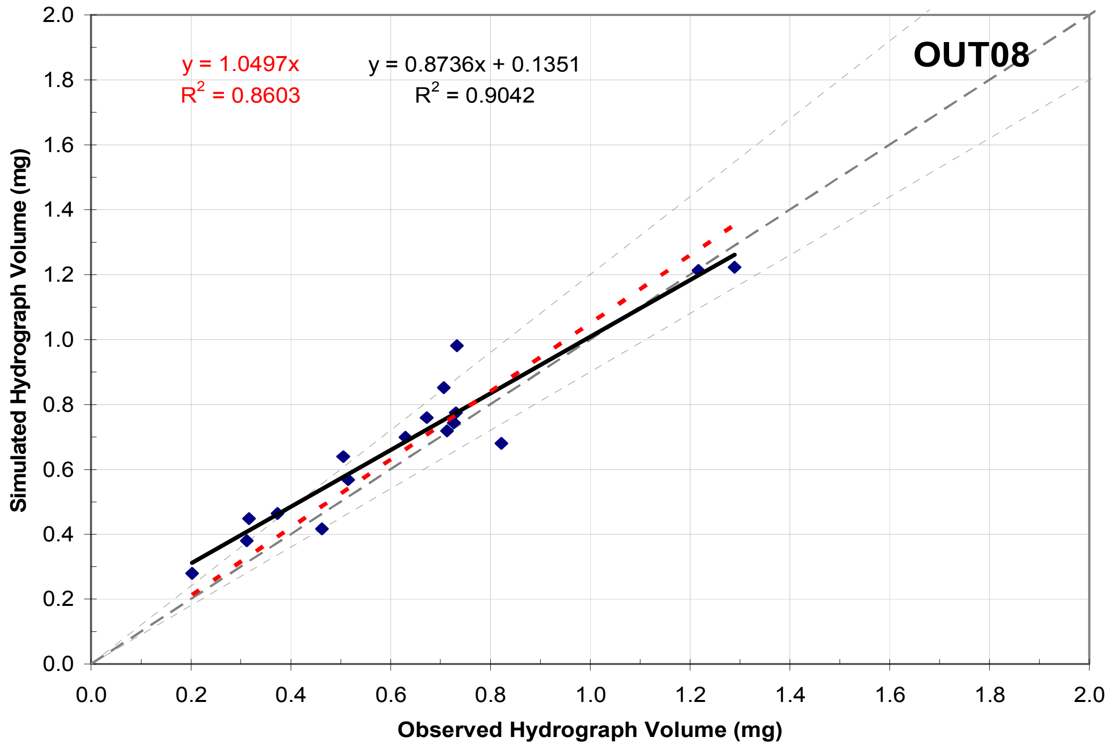


Figure 5.2.3. Statistical Plot of Event Volume for OUT08.

Table 5.2.4 is a very brief summary of the wet weather calibration results for all meter sites. For most of the meters, the average trends of simulated values are within the calibration criteria. For meters OUT04, OUT04A, and OUT06A, the measured values are unrealistically low (compared to neighboring meters) and this is the reason that the simulated values are not within the calibration criteria. For TSOUT01A and TSOUT02 the measured values may be high for peak flow and volume. The simulated values at these sites are very sensitive to the assumed water level boundary conditions used at the Baltimore County line (as discussed above in the dry weather calibration discussion).

For meter HL03, the simulated peak flow values are much higher than the measured peak flows at HL03, but they are consistent with peak measured peak flows upstream at meter HL04. High water levels in the Outfall Sewer cause surcharging at meter HL03, likely were causing the measured peak flow values to be unrealistically low.

Table 5.2.4 - Summary of Wet Weather Calibration

Metershed	Peak Depth	Peak Flow	Volume
HL01	OK	OK	OK
HL02	OK	OK	OK
HL02B	OK	OK	OK
HL03	OK	Measured Values Low	OK
HL04	OK	OK	OK

Table 5.2.4 - Summary of Wet Weather Calibration			
Metershed	Peak Depth	Peak Flow	Volume
HL05	OK	OK	OK
OUT01	OK	OK	OK
OUT02	OK	OK	OK
OUT03	OK	OK	OK
OUT04	OK	Measured Values Low	Measured Values Low
OUT04A	OK	Measured Values Low	Measured Values Low
TSHL01	OK	OK	OK
OUT05	N/A	N/A	N/A
OUT06	OK	OK	OK
OUT06A	OK	Measured Values Low	Measured Values Low
OUT07	OK	OK	OK
OUT08	OK	OK	OK
OUT09	OK	OK	OK
TSOUT02	Measured Values Low	Measured Values High	Measured Values High
TSOUT01A	OK	Measured Values High	Measured Values High
TSOUT01B	OK	OK	OK

5.3 Baseline Analysis and Capacity Assessment

The Baseline Analysis and Capacity Assessment (BACA) is an evaluation of the hydraulic performance of the sewer network in the Outfall Sewershed for baseline and future 2025 conditions during dry weather and wet weather conditions. Guidelines and requirements for the BACA are provided in the Baltimore Sewer Evaluation Standards (BaSES) Manual Sections 7.6.3 and 7.8.2. The analysis is based on simulated hydraulic model results that assess the capacity of the system for baseline and future conditions for a variety of design storms.

Baseline conditions are comprised of the existing sewer infrastructure and flow based on the 2007 population and land use conditions. Future 2025 conditions in the Outfall Sewershed have the same sewer network as the Baseline conditions except for the disconnection of the 15-inch pipe serving meter basin OUT05. The City performed a project in 2009 to connect this 15-inch pipe to the Low Level Sewershed. Subcatchment flows from the Outfall Sewershed are based on estimates of the future 2025 population and land use; this produces an 8.5% increase from the baseline base sanitary flow rates. The degradation of the sewer system is modeled as a 10% increase in the groundwater infiltration rate. Other features of the Baseline model remain the same in the Future 2025 model, such as the wet weather flow characteristics of the subcatchments and the sediment levels.

Subsequent to the writing of the BACA report, the Future 2025 boundary conditions were revised to account for proposed conveyance system improvements in the upstream sewersheds. This set of boundary conditions are designated “Upstream Improvements” and the results of the future capacity analysis using these boundary conditions is reported in the AARR and summarized in this Sewershed Plan.

Attachment 5.3.1 is the BACA report and its associated appendix documents.

5.3.1. Design Storms

Rainfall depths related to specific design storms are published by the National Weather Service (1). The time varying rainfall patterns (hyetographs) of the design storms were defined by the technical program manager based on the NRCS/NOAA rainfall distribution (2). Table 5.3.1 lists the design storms to be used for the BACA. These events are defined in BaSES Manual Section 7.6.1 and fulfill the requirements of CD paragraph 9 F ii.

Table 5.3.1 - Rainfall Design Storms

Rainfall Recurrence Interval	Rainfall Duration	Rainfall Depth (inches)
3-month	1 hour*	1.11
1-year	24 hour	2.67
2-year	24 hour	3.23
5-year	24 hour	4.15
10-year	24 hour	4.97
15-year	24 hour	5.41
20-year	24 hour	5.82

* Approximately equal to the time of concentration of the Outfall Sewershed subcatchments.

Figure 5.3.1.1 shows the rainfall hyetographs in which the peak rainfall intensity starts just after noon in the middle of the 24-hour duration.

This distribution is intended to represent an event that has the same rainfall recurrence interval for several duration periods. For example, Figure 5.3.1.2 shows the depth-duration-frequency relationship for the 5-year recurrence interval event. The rainfall depth satisfies the 5-year recurrence interval not only for the overall 24-hour duration, but also for 1, 3, 6, and 12 hour durations. The benefit of this type of rainfall distribution is that subcatchments of various sizes (and various times of concentration values) experience a rainfall input that has an equal frequency of recurrence.

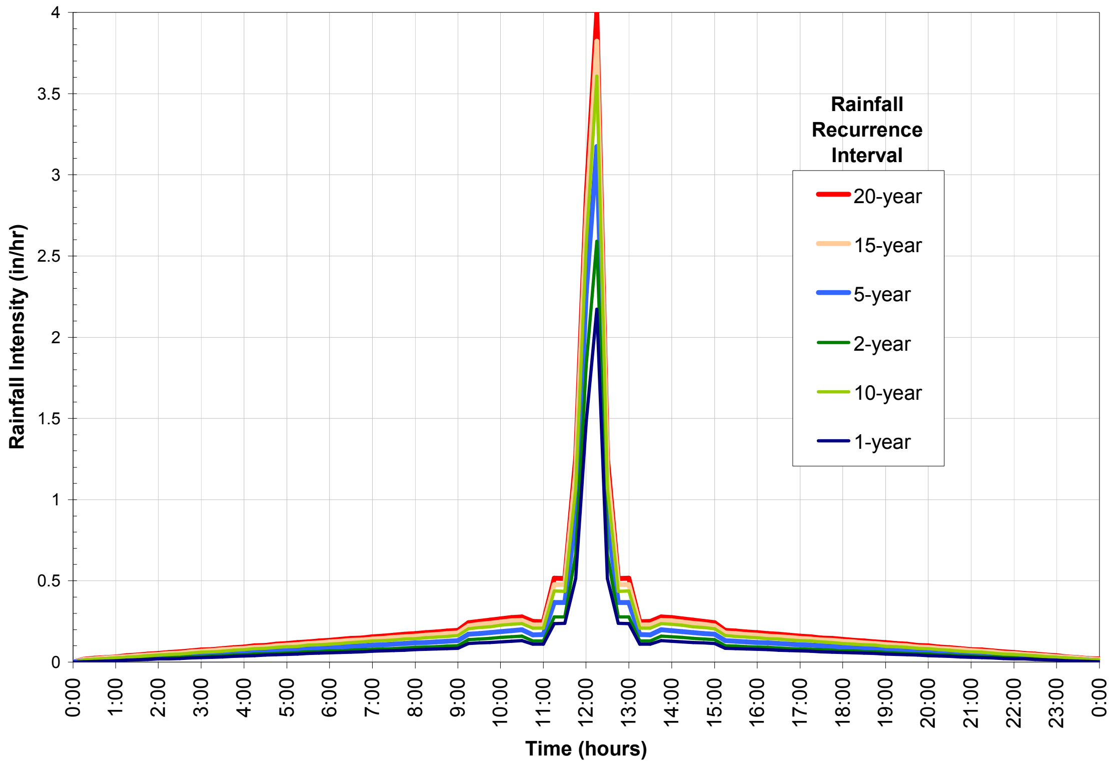


Figure 5.3.1.1: Design Storm Rainfall Hyetographs

5.3.2. Definition of Deficiency

Capacity is defined in the BaSES manual, Section 7.6, as the level of service which the system can provide without an overflow. Surcharging is allowed as long as water levels do not exceed manhole rim elevations.

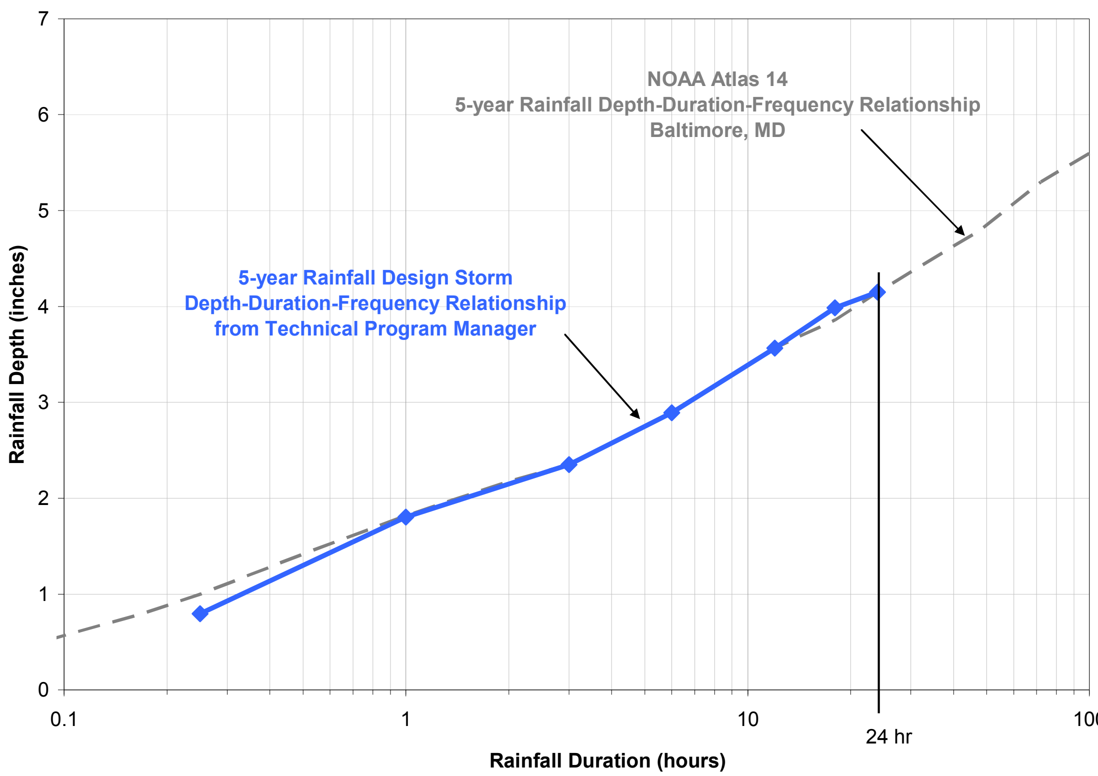


Figure 5.3.1.2: Design Storm Rainfall Depth-Duration-Frequency Relationship

5.3.3. Storm Simulations (All Storms)

This section summarizes the risk of overflows in the Outfall Sewershed by identifying SSO locations and quantifying the SSO risk in terms of simulated SSO volume for the various design storms. The next section, 5.3.4-Identification of Hydraulic Deficiencies, identifies the pipe segments that do not have adequate conveyance capacity. These pipes contribute to the cause of the SSOs.

The BACA evaluation of the Outfall Sewershed uses two alternative scenarios:

- The Active Boundary Conditions scenario incorporates upstream sewersheds flows into the Outfall Sewershed model and downstream water levels at the Baltimore County line.
- The Inactive Boundary Conditions scenario assumes no input flow from the upstream sewersheds and a free flowing outlet at the downstream boundaries of the Outfall Sewershed model. In this scenario the branch sewers have a free discharge into the trunk sewers, which is necessary to identify overflows in the branch sewers that are caused by hydraulic restrictions in the branch sewers themselves.

The results of the active boundary conditions simulations are strongly influenced by the downstream water level boundary conditions at the Baltimore County line, which in turn depend on constraints at the Back River Wastewater Treatment Plant (WWTP).

Maps identifying the locations of overflows for all seven design storm simulations are available in the appendix of the BACA report (Attachment 5.3.1). The maps indentify overflow locations for Baseline (Year 2007) and Future Year 2025 conditions using Active and Inactive boundary conditions.

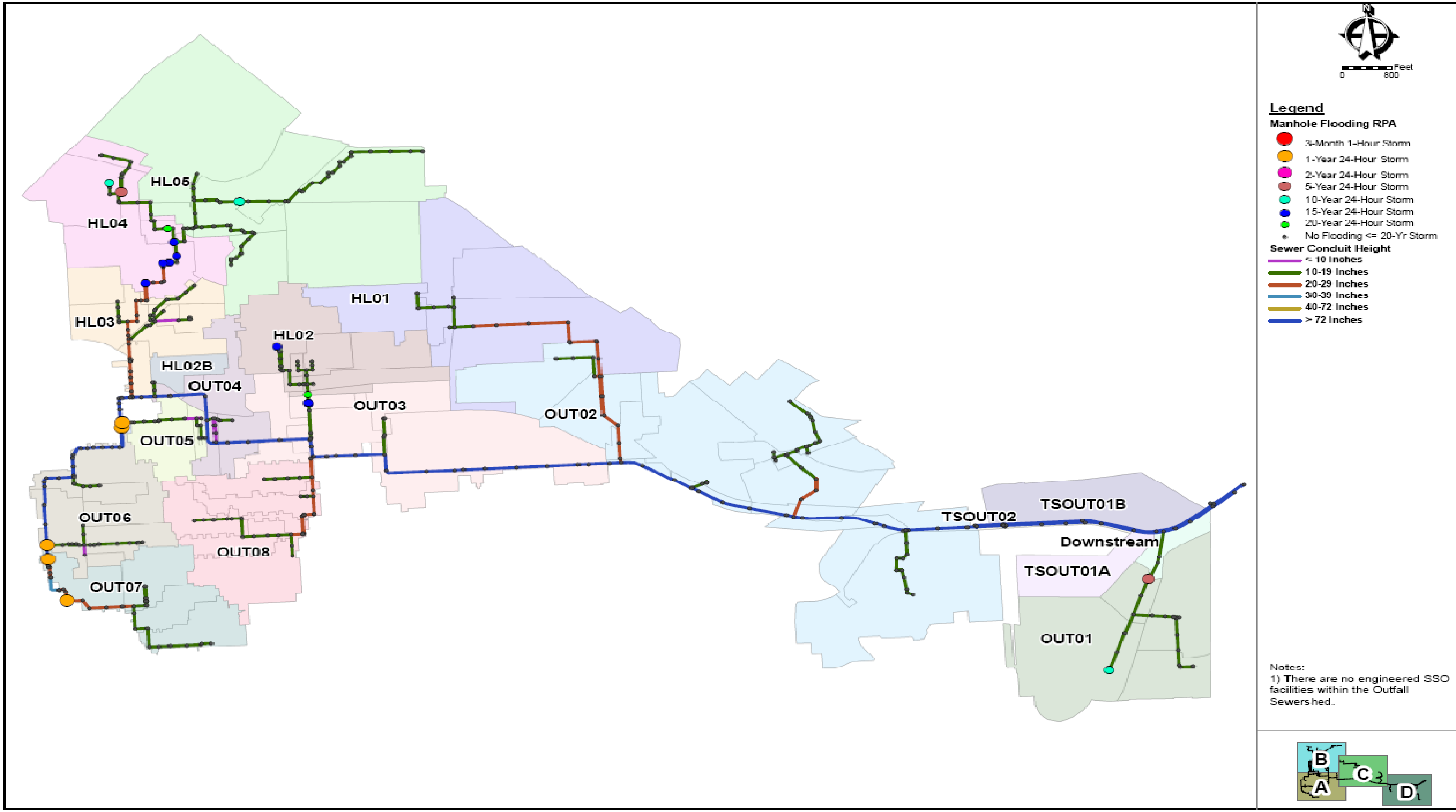
Map 5.3.3 is one of the maps from Attachment 5.3.1 showing locations with simulated SSOs and locations with a risk of SSOs; this particular map is for Baseline Active Boundary Conditions.

Three manholes have the largest SSO volumes among those in Outfall Sewershed. The SSO volumes at the other manholes are relatively small compared to the largest three. Table 5.3.2 lists the SSO volumes at each manhole. The order of the list is ranked from the largest to the smallest volumes. The table also lists the manholes with simulated maximum HGLs within 4 inches of the ground surface; there is no simulated SSO volume at these manholes, they are simply marked as locations where there is a risk of an SSO.

The largest SSO volume is at manhole S45CC_007MH (Durham Street, south of Eager Street) on the 99-inch sewer. This manhole is near the downstream end of the 99-inch sewer, just upstream of the 15-inch connection from OUT05. The second largest SSO volume is simulated at manhole S45CC_021MH (Eager Street, at Durham Street), on the 15-inch sewer serving OUT05, adjacent to the connection with the 99-inch sewer. These two manholes account for approximately 90 to 95% of the total SSO volume in the Outfall Sewershed. (In Future 2025 conditions, the 15-inch Eager Street sewer is disconnected from the 99-inch sewer and is tributary to the Low Level sewershed.)

The third largest SSO volume is at S43E__016MH (Bethel Street and Moyer Street) along the 24-inch branch sewer that serves OUT07. This location is subject to back flow conditions from high water levels in the 99-inch trunk sewer that are strongly influenced by the operations of the Eastern Avenue Pumping Station. The overflows at this manhole account for 5 to 7% of the total SSO volume in the Outfall Sewershed.

Closely associated with the SSO at S43E__016MH is a much smaller SSO volume from the 99-inch sewer at manhole S43A__038MH (Bond Street at Orleans Street). The volume of overflow at Orleans Street is roughly 2% of the volume at Bethel Street. Both overflow locations provide relief to the system near the upstream end of the 99-inch sewer and are driven by high pumping rates from the Eastern Avenue Pump Station.



Map 5.3.3 Simulated Overflow Locations for Baseline Conditions

At the time of writing the BACA report, the boundary conditions to be applied to the Outfall Sewershed model for the Future 2025 conditions did not reflect improvements to facilities in upstream sewersheds that had the potential to increase flows to the Outfall Sewershed. Subsequent to that time, the Future 2025 boundary conditions were refined by the Technical Program Manager to reflect the recommended upstream improvements. The Future 2025 results in the original BACA report are useful in that they identify locations with a SSO risk and sections of pipes that have hydraulic restrictions. The qualitative results are informative, but the numerical magnitude of overflow volumes and peak overflow rates in the original BACA report for the Future 2025 condition are based on the original boundary conditions which produce significantly smaller simulated overflows.

Simulation results with revised boundary conditions were presented in the Alternatives Analysis and Recommendations Report (AARR). The revised results are referred to as simulations with the “Upstream Improvements” boundary conditions. As a result of these planned upstream improvements, flow hydrographs from the Low Level and High Level sewersheds are significantly larger (in volume and peak flow rate) and the downstream level boundary conditions at the County Line are significantly higher. The simulated overflow volumes are listed in Table 5.3.3. The overflow volumes with Upstream Improvements are the basis for evaluating the performance of alternatives in the next section.

Table 5.3.2 - SSO Volume – Baseline Flooding Return Period Analysis – Active Boundary Conditions								
	Manhole SSO Volume (MG)							
	Rainfall Return Period							
Manhole	**DWF**	**3 mo**	**1 yr**	**2 yr**	**5 yr**	**10 yr**	**15 yr**	**20 yr**
S45CC_007MH			0.952	3.262	6.830	9.734	12.027	13.970
S45CC_021MH			1.812	2.865	3.934	4.651	5.002	5.278
S43E__016MH			0.140	0.396	0.903	1.284	1.327	1.593
S43A__038MH[1]			0.0002	0.0002	0.028	0.070	0.021	0.037
S43C__022MH			risk	risk	risk	risk	risk	risk
S69C__002MH					0.001	0.054	0.088	0.118
S45OO_014MH					0.010	0.037	0.051	0.061
S69G__005MH						0.023	0.051	0.081
S47MM_042MH						0.017	0.041	0.066
S43OO_002MH						0.001	0.006	0.012
S45KK_020MH							risk	risk
S45KK_031MH							0.006	0.016
S49EE_004MH							0.002	0.011
S45KK_026MH							0.001	0.004
S45KK_003MH							0.001	0.004
S49GG_039MH							0.0002	0.007
S45MM_014MH							risk	0.001
S49EE_007MH							risk	risk
S49EE_029MH								0.001
S45MM_002MH								risk
S45MM_018MH								risk
Total SSO			**2.9**	**6.5**	**11.7**	**15.9**	**18.6**	**21.3**

[1]Overflow volume at manhole S43A__038MH is associated with overflow at S43E_016MH.
[2] "Risk" means the simulated water level is within 4 inches of the manhole rim.

Table 5.3.3 - SSO Volume – Future 2025 Flooding Return Period Analysis – Upstream Improvements Conditions							
Manhole	**2-yr**	**5-yr**	**10-yr**	**15-yr**	**20-yr**	**Meter Basin**	**Location**
S45CC_007MH	23.137	33.544	40.447	44.689	46.721	OUT06	Durham Street, south of Eager Street
S45CC_021MH	-	-	-	-	-	OUT05	Eager Street, at Durham Street (Future: Disconnected from Outfall)
S43E__016MH	1.487	2.115	2.595	2.914	3.125	OUT07	Bethel Street and Moyer Street
S43A__038MH	1.275	2.742	3.851	4.481	4.926	OUT06	Bond Street, at Orleans Street
S43C__022MH	0.206	0.741	1.411	1.683	1.983	OUT06	Bond Street, between Orleans Street and Fayette Street
S69C__002MH	0.000	0.003	0.095	0.145	0.189	OUT01	Sewer along RR tracks parallel to and between Kane St and Interstate 95. Behind the City of Baltimore Solid Waste Station at 111 Kane St.
S45OO_014MH	0.000	0.010	0.037	0.050	0.061	HL04	Wolfe Street at Darley Avenue
S69G__005MH	0.000	0.000	0.025	0.053	0.084	OUT01	Railroad tracks between Kane St and Interstate 95, at Eastern Ave.
S47MM_042MH	0.000	0.000	0.017	0.040	0.065	HL05	Sinclair Lane at Homestead Street
S43OO_002MH	0.000	0.000	0.001	0.006	0.012	HL04	Cliftview Avenue, half a block east of Wolfe Street
S45EE_015MH	-	-	-	-	-	near OUT06	Durham Street, south of Chase Street
S45KK_020MH	0.000	0.000	0.000	0.000	0.000	HL04	Lanvale Street, where the sewer turns south along Washington Street
S45KK_031MH	0.000	0.000	0.000	0.007	0.017	HL04	Lafayette Avenue, where the sewer turns south along Castle Street
S49EE_004MH	0.000	0.000	0.000	0.004	0.015	HL02	Luzerne Avenue, at Beryl Avenue
S45KK_026MH	0.000	0.000	0.000	0.002	0.005	HL04	Lafayette Avenue, between Chester Street and Castle Street
S45KK_003MH	0.000	0.000	0.000	0.001	0.005	HL04	Chester Street (west side of street), north of Lafayette Avenue
S49GG_039MH	0.000	0.000	0.000	0.001	0.008	HL02	Milton Avenue, north of Preston Street
S45MM_014MH	0.000	0.000	0.000	0.000	0.001	HL04	Chester Street (east side of street), south of North Avenue
S49EE_007MH	-	-	-	-	-	HL02	Luzerne Avenue, at Beryl Avenue
S49EE_029MH	0.000	0.000	0.000	0.000	0.001	HL02	Luzerne Avenue, between Beryl Avenue and Chase Street
S45MM_002MH	-	-	-	-	-	HL04	Alley parallel to North Avenue and E. 20th Street, between Castle Street and Chester Street
S45MM_018MH	-	-	-	-	-	HL04	Chester Street (west side of street), south of North Avenue
S49GG_032MH	-	-	-	-	-	HL02	Biddle Street, just east of Luzern Avenue
S43C__017MH	0.000	0.000	0.004	0.000	0.014	OUT07	just south of Fayette and Bond
S43C__026MH	0.000	0.000	0.003	0.000	0.011	OUT07	just south of Fayette and Bond
Sum of SSO (MG)	**26.1**	**39.2**	**48.5**	**54.1**	**57.2**		**Total for the Outfall Sewershed only**
S43EE_034MH	3.2	6.2	8.8	9.7	11.2	HL end	High Level Sewershed, Chase near Rutland, just upstream of the Outfall Interceptor
Sum of SSO (MG)	**29.3**	**45.4**	**57.3**	**63.8**	**68.5**		**Total including overflow in High Level at S43EE_034MH**

5.3.4. Identification of Hydraulic Deficiencies (All Storms)

Hydraulic deficiencies are sections of pipe that do not have adequate conveyance capacity; these are also called hydraulic restrictions in the BACA report.

Sediment accumulations in the large trunk sewers (99-inch pipe, Outfall Interceptor, and Outfall Relief sewer) reduce the conveyance capacity. The capacities of the large trunk sewers are not sufficient to convey the peak flows. Even without sediment, the capacities of the large trunk sewers are not sufficient to convey the peak flows used in the simulations.

In certain critical locations, the sewer system within the Outfall Sewershed has little tolerance for surcharging. Several manholes in the vicinity of the junction at the upstream end of the Outfall Interceptor (Chase and Durham Streets) have low ground surface elevations. Manhole S45CC_021 MH (Eager Street, at Durham Street) on the 15-inch pipe from OUT05 has the lowest ground surface elevation in this area. Only 1.8 feet of surcharge at the upstream end of the Outfall Interceptor is possible before manhole S45CC_021MH starts to overflow. Other manholes on the 99-inch sewer and the 100-inch sewer from the High Level Sewershed are also shallow and are at risk of SSOs.

The 15-inch sewer from OUT05 was disconnected from the 99-inch sewer in 2009 and this change is reflected in the Future 2025 model setup. The disconnection eliminates the SSO at manhole S45CC_021MH, but increases the volume of SSO at a nearby manhole on the 99-inch sewer (manhole S45CC_007MH, Durham Street, just south of Eager Street). The volume of flow in the 15-inch pipe is relatively small. The model accounts for the fact the tributary area to the 15-inch pipe no longer contributes flow to the 99-inch pipe, but the impact of this disconnection on the overall hydraulic performance of the Outfall Sewershed is negligible.

Meter basin OUT07 is served by a branch sewer that connects to the upstream end of the 99-inch sewer. Manhole S43E__016MH (Bethel and Moyer Streets) on the 24-inch branch sewer in OUT07 is vulnerable to overflows when the Eastern Avenue Pumping Station is pumping with more than three pumps online. The simulation results show flow reversing in the OUT07 branch sewer when high water levels in the 99-inch sewer are partially relieved by overflowing at manhole S43E__016MH (Bethel and Moyer Streets). Flow meter OUT09 monitors the same branch sewer as meter OUT07. The flow reversal behavior is observable in the raw 5-minute data for OUT09 in the large wet weather event of 11-16-2006.

The branch sewer from OUT01 is vulnerable to overflows at manhole S69C__002MH in the 5-year event because of relatively high flows and a low ground elevation. Manhole S69C__002MH is on the 18-inch sewer along the railroad tracks between Kane Street and Interstate highway 95 (behind the City of Baltimore Solid Waste Station at 111 Kane Street).

Near the upstream end of the HL04 meter basin, manhole S45OO_014MH (Wolfe Street at Darley Avenue) has a simulated SSO in the 5-year event. The low ground elevation over the pipe (cover less than 4 feet) makes this manhole vulnerable to overflows. The overflow is caused by a hydraulic restriction in the 10-inch pipe along Wolfe Street between Darley Avenue and Sinclair Lane.

Manhole S47MM_042MH (Sinclair Lane at Homestead Street) is the location of a SSO for the 10-year event in the HL05 meter basin. The ground surface elevation is approximately 6 feet lower than other manholes along Sinclair Lane. The hydraulic restriction in the 12-inch Collington Avenue line contributes significantly to the cause of this SSO.

The remaining SSO locations are associated with infrequent return period events and high flows all along the length of the branch sewers rather than localized hydraulic restrictions. High water levels in the Outfall sewer in the active boundary condition scenario contribute to the occurrence and severity of these overflows, but the overflow volumes are relatively small.

5.4 Alternative Analysis (2-Year and Larger Storms)

The Alternative Analysis and Recommendation Report (AARR) is a discussion of the development and evaluation of facilities for three alternatives that eliminate SSOs in the Outfall Sewershed. The objectives of the Consent Decree relevant to the AARR are defined in the BaSES Manual, particularly sections 7.7, 7.8.3, and 8.2. The alternatives mitigate SSOs for design storms of increasing severity. Attachment 5.4.1 contains the AARR.

The Outfall Sewershed is unique among all of the Baltimore sewersheds in that most of the flows conveyed through the Outfall Sewershed network originate from upstream sewersheds (Jones Falls, High Level, Low Level, Herring Run, and Dundalk). A relatively small fraction of the flow originates from the subcatchment areas within the Outfall Sewershed. Consequently, the largest and most costly alternative facilities are sized to accommodate the high flows from upstream sewersheds. Conveyance improvements in the upstream sewersheds have the potential to increase the risk of SSOs in the Outfall Sewershed and have a direct influence on the size and cost of the required alternative facilities.

All of the alternatives assume that sediment is removed from the 99-inch sewer, Outfall Interceptor, and Outfall Relief sewer. Sediment removal increases the conveyance capacity by restoring the full cross section area and reducing the hydraulic roughness of pipes.

Alternative 1 proposes two storage tanks, one at Fayette and Bond Streets and the other at Chase and Durham Streets, to attenuate the upstream peak flows. Excess flows enter the storage tanks so that the remaining flows are within the conveyance capacities of the pipes. Alternative 1 does not assume any changes downstream at the Back River WWTP.

Alternative 2 assumes that downstream improvements are in place. These improvements must increase the capacity of the Back River WWTP to receive more flow (by either additional treatment capacity or storage at the plant). Downstream improvements greatly increase the conveyance capacity of the Outfall Interceptor and reduce the volume of storage required at upstream locations in the Outfall Sewershed. As a result no storage is needed for the 2-year event and only one storage tank is needed for the 5, 10, 15, and 20-year events. The tank is located at the Fayette relief site and is much smaller than the size of the tanks used in Alternative 1 for the various storms.

Alternative 3 also assumes that downstream improvements are in place. Alternative 3 uses a tunnel from the proposed Fayette Street relief point to a proposed reconnection point along Lombard Street near to the connection from the Dundalk Sewershed.

For the purpose of this study, the downstream improvements are represented in the Outfall Sewershed model as a downstream level boundary condition at the County Line that does not exceed 48 feet (above NAVD88 datum). At 48 feet the Outfall Interceptor and Outfall Relief sewer are approximately 90% full with the water levels one foot below the crowns of the pipes.

While cleaning the sediment from the pipes helps to restore needed conveyance capacity, the peak upstream flows are anticipated to exceed the conveyance capacity of the clean pipes. The limiting hydraulic feature is the Outfall Interceptor from its upstream end to where the Outfall Relief Sewer Starts. Once the Outfall Relief Sewer runs parallel to the Outfall Interceptor there is sufficient capacity to convey simulated wet weather flows. The existing conditions at the BRWWTP are an additional limitation on the peak flow that can be conveyed by the Outfall Interceptor. Therefore, sediment cleaning is not a stand alone solution to the cause of overflows in the Outfall Sewershed.

The AARR contains results of a sensitivity study that examines the risk of failing to achieve the desired level of protection against overflows due to variations in key modeling parameters. In particular, the study evaluated the sensitivity to hydraulic roughness (Manning's n value) of the pipes after they are cleaned of sediment and sensitivity to the operations of the Eastern Avenue Pump Station (EAPS) during wet weather events. When sediment is removed, the Manning's roughness coefficient (n) is assumed to be 0.013. However, because the results are very sensitive to this assumption, the system performance was also evaluated for a Manning's roughness value of 0.015 to determine the necessary facilities to perform adequately for sub-optimum conditions. The results of the sensitivity analysis are in the AARR.

5.4.1 Description of Trunk Sewer Alternatives

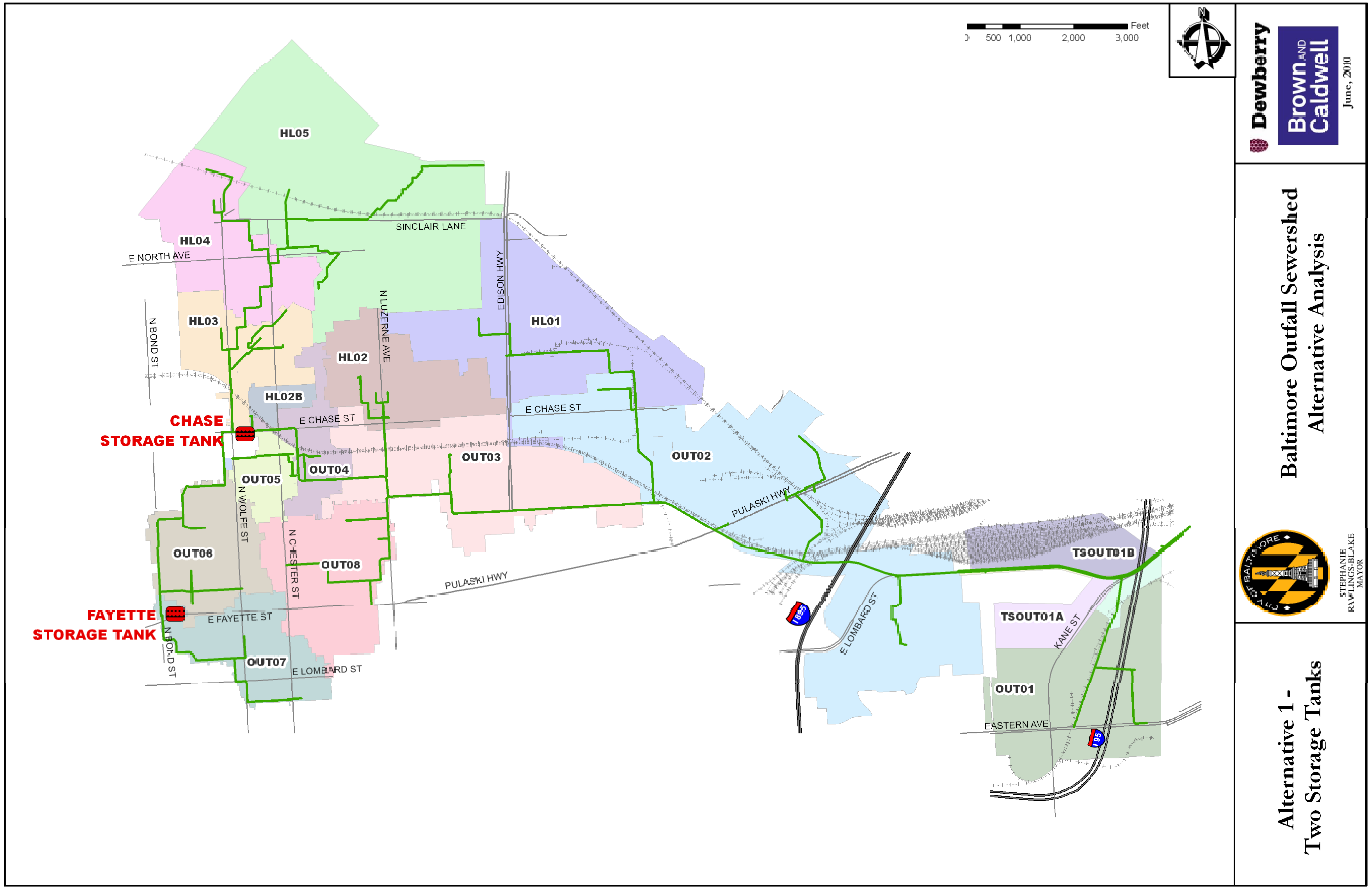
It is assumed that sediment is removed from the trunk sewer in all of the alternatives presented below.

Alternative 1: Storage Using Two Tanks

Alternative 1 uses two storage tanks to store excess flow and prevent SSOs as shown in Map 5.4.1.1. An overflow weir at the upstream end of the 99-inch sewer is needed in the vicinity of Bond and Fayette Streets. This relief facility is called the Fayette weir in the discussion below. The facility should be located between Fayette and Orleans Streets, in close proximity to the connection from the Eastern Avenue Pump Station force main. The purpose of the Fayette weir is to limit the maximum water level at the upstream end of the 99-inch sewer to approximately 58 feet; at this level the 99-inch sewer is surcharged 3 feet and the risk of a SSO further upstream along the 24-inch branch sewer at Bethel and Moyer Streets (manhole S43E__016MH) is minimized.

Relief is also needed to protect the upstream end of the Outfall Interceptor from excessive surcharging in the vicinity of Chase and Durham Streets. The purpose of the Chase weir is to limit the maximum water level at the upstream end of the Outfall Interceptor to no more than 57 feet; at this level the Outfall Interceptor is surcharged 3 feet and the risk of an SSO is reduced at Durham and Eager Streets (manhole S45CC_007MH). The Chase weir should be relatively long to allow significant overflow rates (into a storage tank) with a relatively small head on the weir.

The two storage tanks attenuate the peaks of the inflow hydrographs so that peak flows are within the capacities of the large diameter trunk sewers assuming that the sediment has been removed. Alternative 1 assumes that there are no changes downstream at the Back River WWTP; consequently, the Outfall Interceptor is surcharged to within 2.5 feet of the ground surface at the County Line in the 2-year event. Without improvements at the Back River WWTP, the tanks in this alternative are sized to store the excess flow that can not be conveyed and treated immediately during the event.



Map 5.4.1.1 Alternative 1 Facilities: Two Storage Tanks

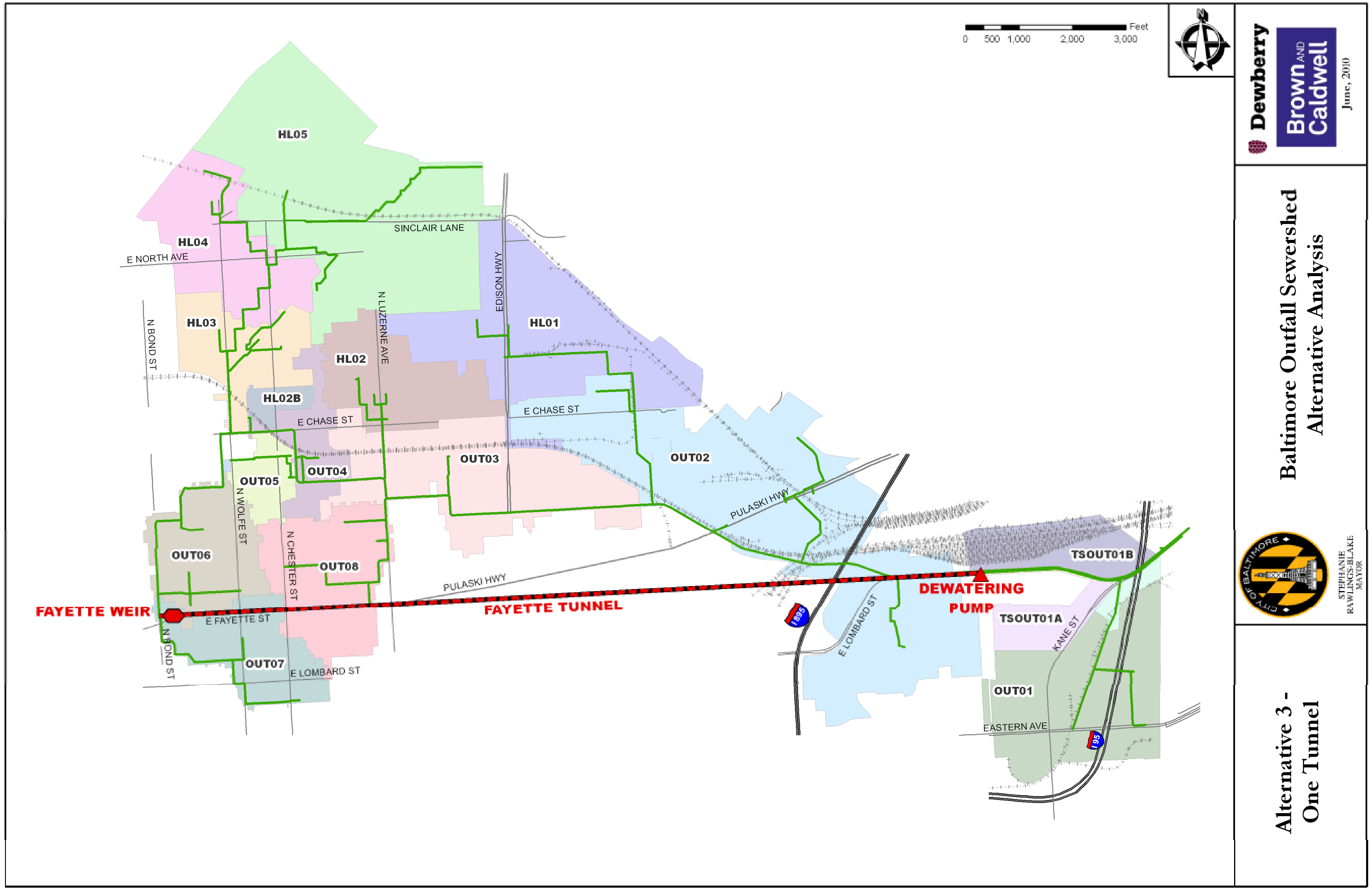
Alternative 2: Storage using One Tank, Assuming Downstream Improvements

Alternative 2 assumes that sediment is removed and downstream improvements at the Back River WWTP will accommodate higher flow rates to the plant. This alternative demonstrates the significant improvement that can be achieved in system performance due to downstream improvements. Assuming that the cleaned pipes have a Manning's roughness value of 0.013, the additional conveyance in the Outfall Interceptor is sufficient to manage the 2-year event without simulated overflows. No new storage at either the Chase or Fayette weir sites is required for the 2-year event. In the larger events, only one storage tank at the Fayette weir location is necessary.

Alternative 3: Storage-Conveyance Tunnel, Assuming Downstream Improvements

Alternative 3 uses a tunnel instead of a storage tank to protect against overflows. The tunnel starts at the Fayette weir location and generally runs in the same west to east direction as the Outfall Interceptor, although it would be a few blocks south. The flow in the tunnel re-enters the Outfall Interceptor along Lombard Street where the Outfall Relief sewer runs parallel to the Outfall Interceptor. In the model, the tunnel connection is near the location where the Dundalk sewer connects to the Outfall Interceptor. Initially, during an event the tunnel provides inline storage volume. After filling and surcharging, the tunnel flows like an inverted siphon to convey flow to the downstream connection point. After an event, the tunnel would be dewatered by a small pump. The tunnel can be seen as an upstream extension of the Outfall Relief sewer. Instead of running immediately parallel to the Outfall Interceptor, the tunnel extends the relief directly to the Fayette Weir location where relief is needed to protect the 99-inch sewer from high pumping rates from the Eastern Avenue Pump Station. By diverting excess flow into the tunnel at the Fayette weir, both the 99-inch sewer and the Outfall Interceptor are protected from overflows.

A significant benefit of the tunnel alternative is that it provides an alternative, parallel flow path to the existing Outfall Interceptor. In the same way that the Outfall Relief sewer provides supplemental conveyance capacity (and in dry weather, a redundant flow path) to the Outfall Interceptor along Lombard St, a relief tunnel would provide an alternative parallel flow path to the upstream section of the Outfall Interceptor. The upstream section of the Outfall Interceptor is a critical link in the overall conveyance system. A major incident that impairs the conveyance capacity of the existing Outfall Interceptor would have a large impact on the City. Major repairs and rehabilitation of the century-old Outfall Interceptor would be much easier to accommodate with a tunnel to serve as a redundant flow path.



Map 5.4.1.2 Alternative 3 Facilities: Storage/Conveyance Tunnel

Comparison of Alternatives:

Table 5.4.1 presents the required storage volumes at the Fayette and Chase weir locations for Alternative 1, 2 and 3 to provided protection from overflows for the 2, 5, 10, 15, and 20-year return period design storms. These results assume nominal roughness conditions (0.013) after sediment cleaning.

Table 5.4.1
Trunk Sewer SSO Alternatives
Storage Volumes (MG)

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Fayette Weir Storage Tank	3.0	7.0	10.5	12.5	14.1
	Chase Weir Storage Tank	3.3	8.1	12.2	14.5	16.5
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Storage Tank	0	2.1	4.2	5.5	6.5
Alternative 3 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Tunnel Siphon Mode	0	1.6	2.5	3.6	3.6

Figure 5.7 shows the storage volumes required for the various alternatives as a function of the storm return periods. Also shown in the figure is the simulated SSO volume with future conditions and upstream improvements. Alternatives are compared to the SSO volume caused by the upstream improvements (dark blue curve in Figure 5.7).

With sediment remaining in the pipes and no downstream improvements, the required storage volume to prevent SSOs is greater than the initial SSO volume. This case is like Alternative 1 (but with sediment remaining) and is shown by the upper gold colored curve in Figure 5.7.

The storage volume required for Alternative 1 is substantially less than the initial SSO volume because of the removal of sediment (orange curve in Figure 5.7). Sediment removal is particularly helpful in all of the alternatives because more of the flow can be conveyed by the existing trunk sewers and less volume needs to be diverted at the Fayette weir.

Downstream improvements at the Back River WWTP are complimentary to sediment removal. The lower blue curve in Figure 5.7 shows the simulated SSO volume if no new facilities are added to the Outfall sewershed and the only actions are the removal of sediment and the downstream improvements. In this case, there are no simulated overflows for the 2-year event and only 3% of the initial SSO volume remains in the 10-year event.

Alternatives 2 and 3 are only needed for the 5-year and larger events. Alternative 2 requires a tank volume that is relatively small (4.2 MG for the 10-year event). Alternative 3 requires a tunnel volume that is even smaller (2.5 MG for the 10-year event in the form of a 5-foot diameter tunnel).

The results shown in Figure 5.4.1.3 emphasize the effectiveness of downstream improvements and sediment removal. Most of the initial SSO volume is removed with those two technologies. A storage tank or a conveyance tunnel is necessary to fully remove the simulated SSO volume and to provide a greater degree of flexibility and robust performance.

The performance of these alternatives is contingent upon adequate treatment capacity at the Back River WWTP. In the Outfall Sewershed model, the assumption of adequate treatment capacity corresponds to elevated peak flow rates at the County Line. Figure 5.4.1.4 is a companion to Figure 5.4.1.3. Figure 5.4.1.3 shows the sum of the peak flows at the County Line (which is the sum of the flow in the Outfall Interceptor and the Outfall Relief sewer). In the baseline simulations, the sum of the peak flows is just less than the existing treatment capacity of 300 MGD. In the alternatives, particularly Alternative 3 with the tunnel, the sum of peak flows is between 400 and 500 MGD at the County Line. This does not include additional flow from Baltimore County. Therefore, these alternatives assume approximately 100 to 200 MGD of additional treatment capacity.

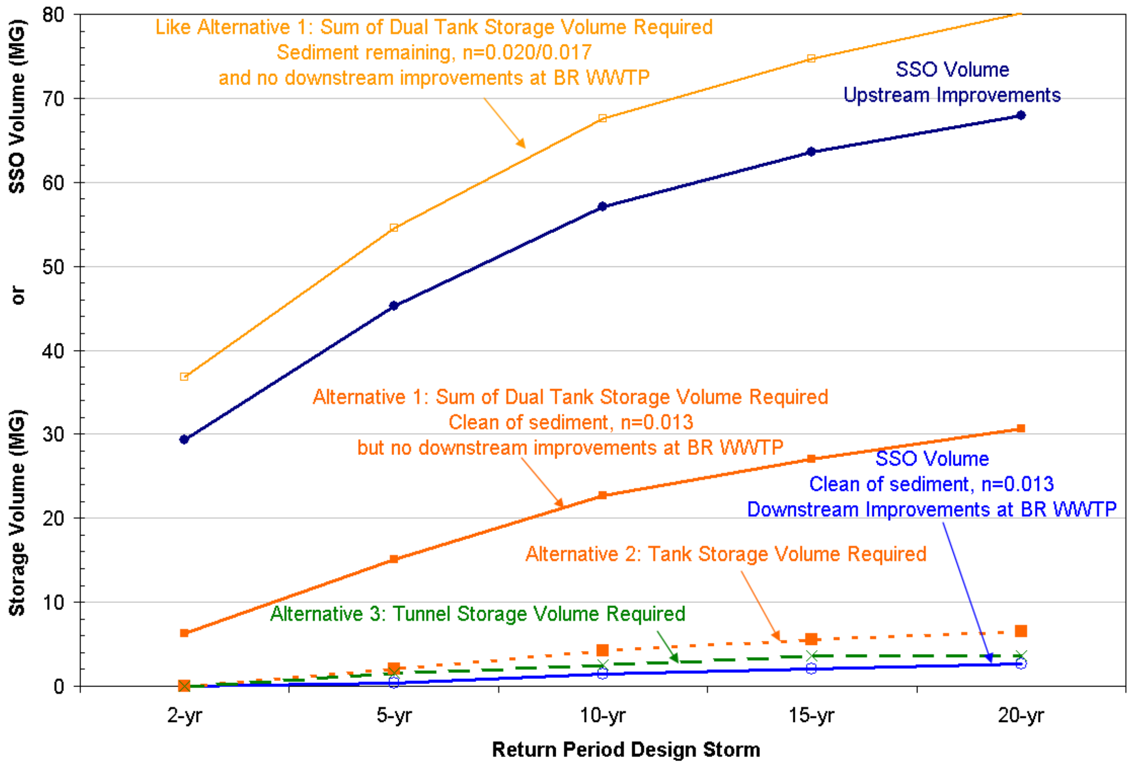


Figure 5.4.1.3 - Alternative Storage Volumes and Baseline SSO Volume

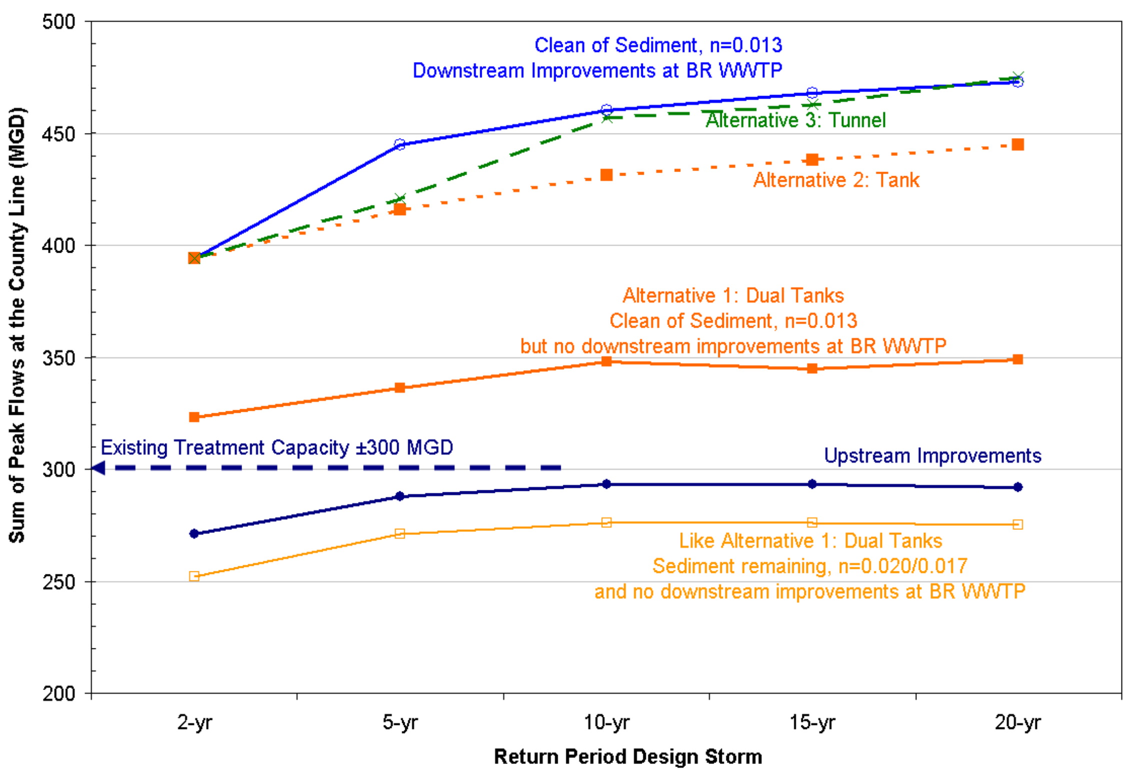


Figure 5.4.1.4 - Sum of Peak Flows at the County Line for Alternatives

Alternative Facilities Evaluated for Sub-Optimal Conditions and Large Wet Weather Events

This section is a discussion of the performance of the 10-year solutions for Alternatives 2 and 3 given above. The 10-year Alternative 2 solution is a 4.2 MG tank. The 10-year Alternative 3 solution is a 5-foot tunnel (2.5 MG volume). The facilities identified for the 10-year event are sized assuming the nominal simulation conditions (roughness of 0.013). In this analysis the performance of the facilities is evaluated for more extreme events and for a higher roughness assumption.

Simulations were run for sub-optimal conditions and larger events to evaluate the robustness of each case. For the purpose of this evaluation, “sub-optimal” conditions are defined to be a Manning’s roughness value of 0.015 and all pumps online at the Eastern Avenue Pump Station.

Figure 5.4.1.5 shows the simulated SSO volume for four cases:

- Upstream Improvements (initial SSO volume)
- Downstream Improvements and Sediment Removed (n=0.015)
- Alternative 2 (4.2 MG storage tank)
- Alternative 3 (5-foot diameter tunnel)

The improvements at the Back River WWTP make the single greatest reduction in SSO volume. Even under sub-optimal conditions, in the 2-year event, only 1% of the SSO volume remains due to the additional capacity of the downstream improvements. In the 20-year event, only 10% of the initial SSO volume remains.

For sub-optimal conditions, Alternative 2 (the 4.2 MG tank) eliminated simulated SSOs for the 2-year event and only 7% of the initial SSO remains in the 20-year event.

For sub-optimal conditions, Alternative 3 (the 5-foot diameter tunnel) eliminated the simulated SSOs for the 2-year event and only 2% of the initial SSO volume remains for the 20-year event. This result assumes that the downstream improvements allow the higher peak flows to be conveyed successfully to the Back River WWTP without surcharging at the County Line.

Both the tank and the tunnel provide significant protection for SSOs in the extreme events (15 and 10-year events), but the tunnel is more effective in minimizing overflows due to its ability to convey excess flow throughout the storm duration. A tunnel would also be more effective than a tank in back-to-back wet weather events because it does not rely on dewatering to restore the functionality of the facility.

These simulation results indicate that a facility sized for a 10-year event with nominal conditions is likely to provided protection against SSOs for a 2-year event in sub-optimal conditions.

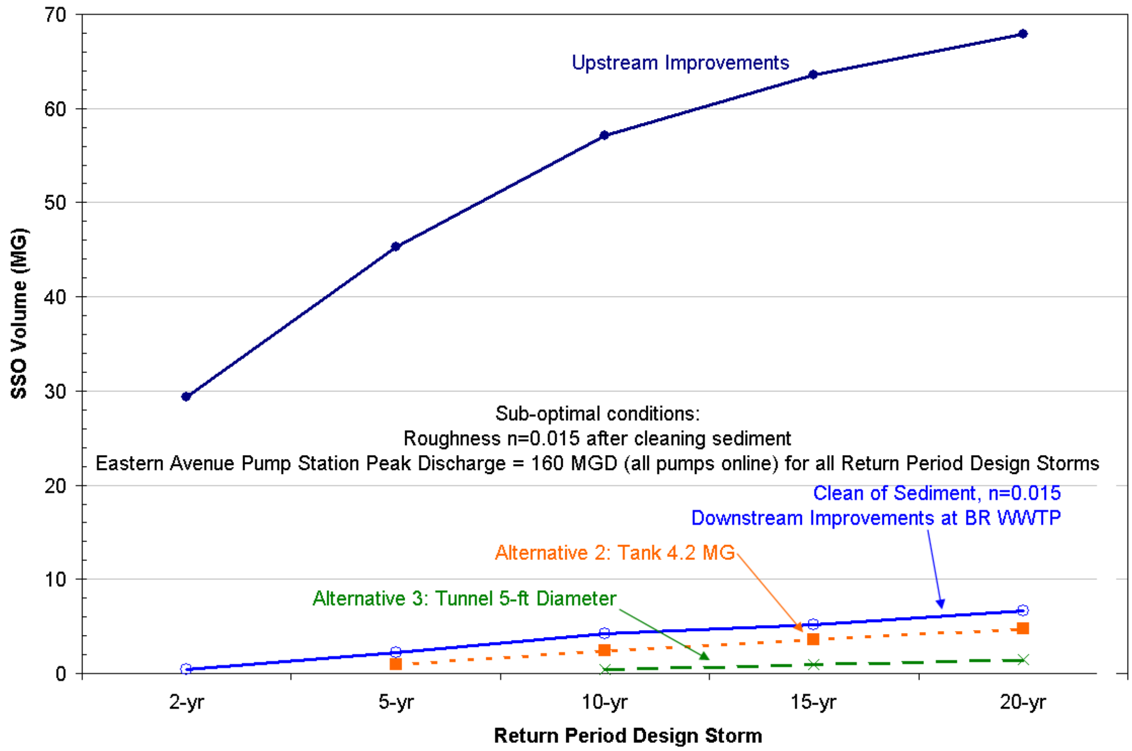


Figure 5.4.1.5 - Simulated SSO Volume for Alternatives in Sub-Optimal Conditions

Figure 5.4.1.6 shows the sum of peak flows at the County Line for the Outfall Interceptor and the Outfall Relief sewer. In the Upstream Improvements simulations, the sum of peak flows is less than 300 MGD in any event. The Outfall Sewershed alternatives assume downstream improvements at the Back River WWTP so that greater flows and lower water levels are possible at the County Line. The alternative simulations assume additional treatment capacity is sufficient to allow the flow at the County Line to increase approximately 100 MGD more than the existing rate in the 2-year event.

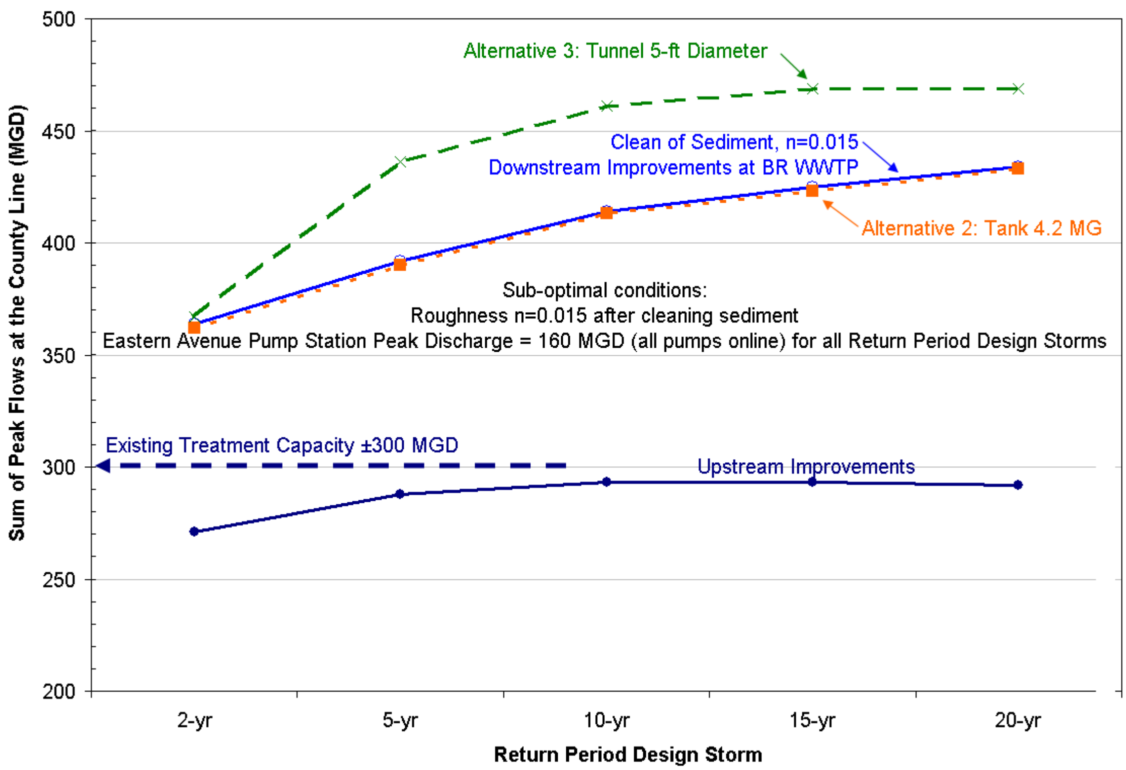


Figure 5.4.1.6 - Sum of Peak Flows at the County Line for Alternatives in Sub-Optimal Conditions

5.4.2 Summary of Improvements

Downstream improvements at the Back River WWTP and the removal of sediment from the sewers are the most effective changes to improve system performance and reduce the likelihood of overflows.

The sewershed plan for the Low Level sewershed does not mention storage to limit peak flows from the Eastern Avenue Pump Station (EAPS). Further investigation is needed using the Macro model to evaluate the trade offs between potential improvements in the Outfall Sewershed and the Low Level Sewershed. Storage upstream of the EAPS would not take advantage of the existing peak discharge capacity of the pump station. Storage located downstream of the EAPS, in the Outfall Sewershed, is likely to be more complimentary to the existing pumping capacity. In essence, the Fayette storage tank alternative serves this role. It remains for the technical program manager, using the Macro model, to further evaluate this topic.

No additional facilities are needed for the 2-year event in the Outfall Sewershed (if the assumed Manning's roughness value is accurate and the Eastern Avenue Pump Station does not operate at full capacity, not exceeding 137 MGD). Even for sub-optimum conditions, the downstream improvements and the removal of sediment are sufficient to remove 99% of the simulated SSO volume in the 2-year event compared to the initial overflow volume with Upstream Improvements.

A moderately sized storage tank or tunnel is needed at the Fayette relief point to fully eliminate SSOs for events greater than the 2-year storm and for sub-optimal conditions. Rather than defining a specific alternative recommendation, the findings of this evaluation and the summary cost tables below are presented for the purpose of discussion with the City. The cost of Alternative 2 (storage tank) is lower than the cost of Alternative 3 (tunnel). Therefore, Alternative 2 is the lowest cost approach to eliminating SSOs in the Outfall Sewershed.

Even though Alternative 3 (tunnel) is not the lowest cost option, it does provide greater flexibility and is more effective in reducing SSO volume for larger events. The advantages of a tunnel include:

- Relief for the 99-inch sewer when the Eastern Avenue Pump Station operates with all pumps on-line
- Effective reduction of SSO volume in extreme events (approximately 1 to 2% of initial SSO volume remaining)
- Functional in back-to-back wet weather events because siphon mode operation does not require dewatering time like a storage tank
- Parallel/redundant flow path to the Outfall Interceptor (useful as a dry weather bypass if the Outfall Interceptor needs maintenance, cleaning, or repair).

The improvements needed for each of the design storms are summarized below for Alternatives 2 and 3 for the nominal conditions. The tables presented in the summaries below itemize the recommended improvements and the costs to implement each improvement. The costs are given for 10 years (which is the span of potential implementation of the projects), from 2008 (the cost “base year”) to 2017, escalated by 7% a year, as required by the methodology described in BaSES Manual, Section 8.3.2.1.

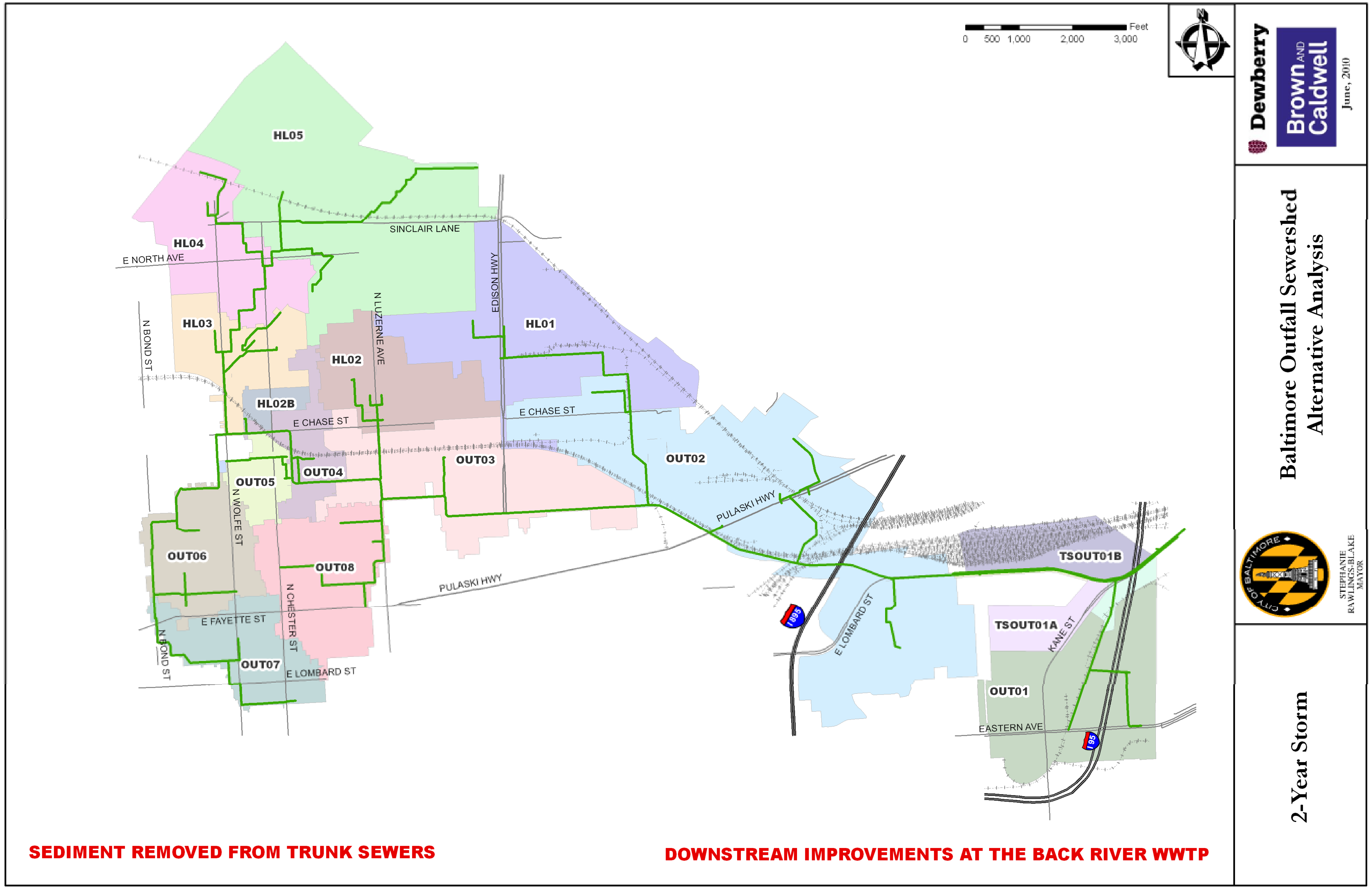
2-Year Improvements

Map 5.4.2.1 shows a summary of the improvements for the 2-year return period event. Sediment cleaning in the large diameter trunk sewers is needed along with downstream improvements.

Costs of the 2-year improvements are itemized in Table 5.4.2; the only cost in the Outfall Sewershed is the cost of removing the sediment. The cost of the downstream improvements is not included in this report. The cost of downstream improvements must include the cost of cleaning of the trunk sewers from the County Line to the Back River WWTP and the cost of storage or capacity upgrades at the treatment plant.

Table 5.4.2
2-year Outfall Improvements
Alternative 3: Sediment Removed

Site	Improvement	Unit Cost	Quantity	Cost
Sediment Cleaning in Trunk Sewers				
99-inch Sewer	Sediment Cleaning	500 $/ton	1,600 tons	$800,000
Outfall Interceptor	Sediment Cleaning	500 $/ton	29,000 tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500 $/ton	3,600 tons	$1,800,000
			Subtotal	**$17,100,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)				$7,182,000
2008 Total Estimated Cost				**$24,282,000**
2009 Total Estimated Cost				**$25,982,000**
2010 Total Estimated Cost				**$27,801,000**
2011 Total Estimated Cost				**$29,747,000**
2012 Total Estimated Cost				**$31,829,000**
2013 Total Estimated Cost				**$34,057,000**
2014 Total Estimated Cost				**$36,441,000**
2015 Total Estimated Cost				**$38,992,000**
2016 Total Estimated Cost				**$41,721,000**
2017 Total Estimated Cost				**$44,641,000**



Map 5.4.2.1 2-year Improvements for Alternative 3

5-Year Improvements

A 4-foot diameter (1.6 MG) tunnel at the Fayette site is needed in the 5-year event along with sediment removal and downstream improvements at the Back River WWTP. Branch sewer improvements are needed in meter basins HL04 and OUT01. The 5-year improvements are shown in Map 5.4.2.2.

Peak flows surcharge the sewers for the entire length of meterbasins HL03 and HL04 from the upstream end (north of Sinclair Lane) to the downstream connection at the Outfall Interceptor (at Wolfe Street and Chase Street). There is a risk of SSOs at several locations along this sewer system where the maximum HGL approaches the ground surface. Overflows are most likely at manhole S45OO_014MH (Wolfe Street and Darley Street) because of a low ground surface elevation at this point (less than 4 feet of cover). The SSO location is active for the 5-year and larger events.

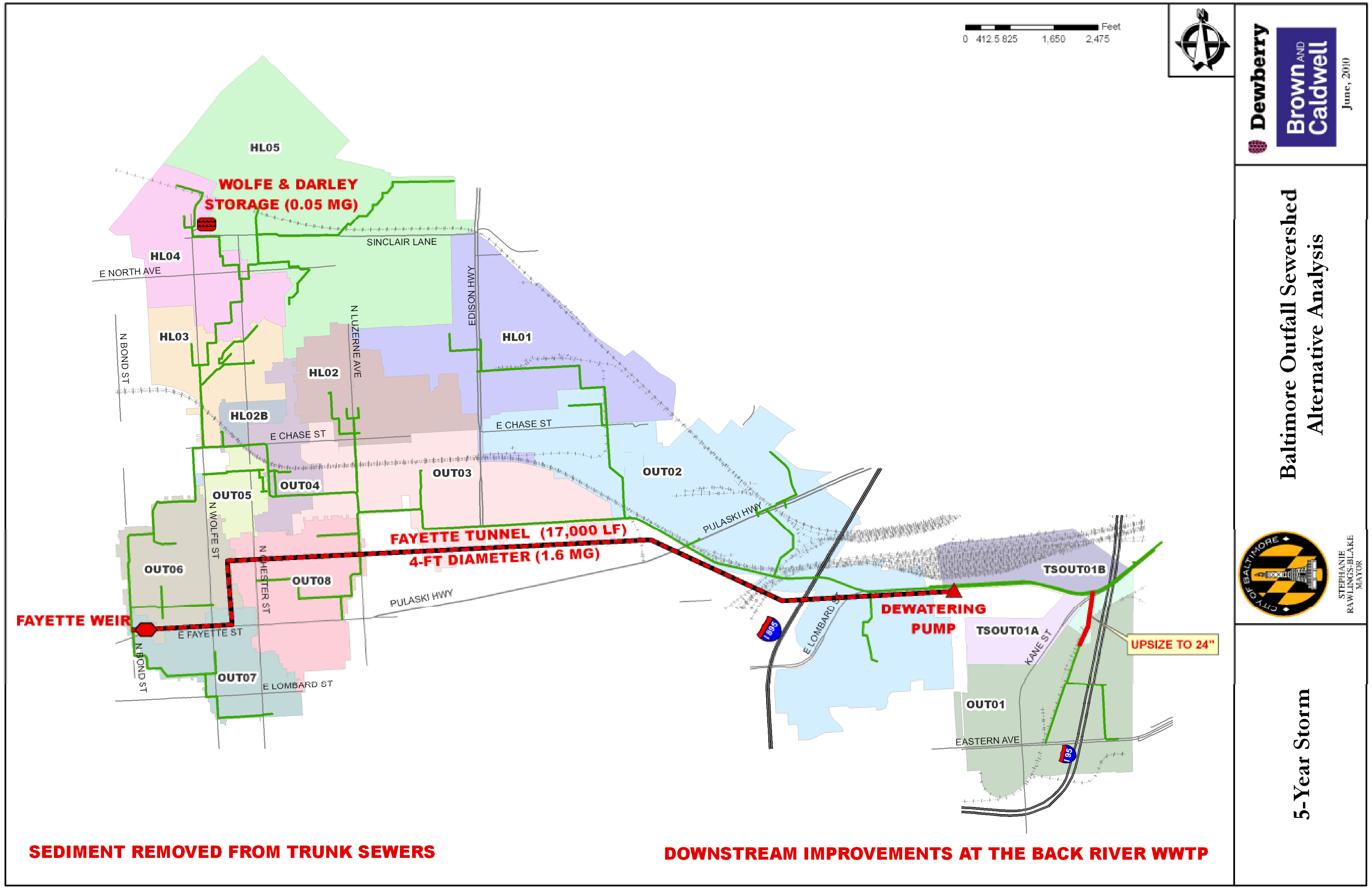
Possible solutions include sealing the manhole, raising the manhole rim to an elevation that is similar to neighboring manholes (approximately 3 feet), building a small storage tank, or rehabilitation of sewers in the Darley/Cliftview Avenue neighborhood to reduce infiltration and inflow (I/I). A storage tank alternative or sewer rehabilitation to reduce I/I will reduce peak flows to the downstream pipes leading to the Outfall Interceptor, thus decreasing the risk of SSO at other locations which do not have simulated SSOs but are at risk of SSOs due to high water levels.

The 18-inch sewer serving meterbasin OUT01 runs along the railroad tracks parallel to and between Kane Street and the Interstate-95 freeway. There is one simulated SSO location in the lower section of the pipe for the 5-year and larger events. The simulated SSO is caused by high simulated peak flows that exceed pipe capacity. The volume of the SSO increases when the Outfall Interceptor is surcharged, but this downstream surcharge condition is not the primary cause of the SSO. The alternative solution is a 24-inch sewer replacement, running 1012 LF from manhole S69C__002MH to the connection to the Outfall Interceptor at manhole S71A__007MH.

Costs of the 5-year improvements are itemized in Table 5.4.3.

Table 5.4.3
5-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.047	MG	$282,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 4' x 17,000 LF	44.14	$/gal	1.6	MG	$70,533,060
	Dewatering Pump	3.00	$/gpd	2	MGD	$6,000,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$95,008,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$39,903,000
2008 Total Estimated Cost						**$134,911,000**
2009 Total Estimated Cost						**$144,355,000**
2010 Total Estimated Cost						**$154,460,000**
2011 Total Estimated Cost						**$165,272,000**
2012 Total Estimated Cost						**$176,841,000**
2013 Total Estimated Cost						**$189,220,000**
2014 Total Estimated Cost						**$202,465,000**
2015 Total Estimated Cost						**$216,638,000**
2016 Total Estimated Cost						**$231,803,000**
2017 Total Estimated Cost						**$248,029,000**



Map 5.4.2.2 5-year Improvements for Alternative 3

10-Year Improvements

For 10-year level of protection, a 5-foot tunnel (2.5 MG) is required at the Fayette relief point. The 10-year improvements are shown in Map 5.4.2.3.

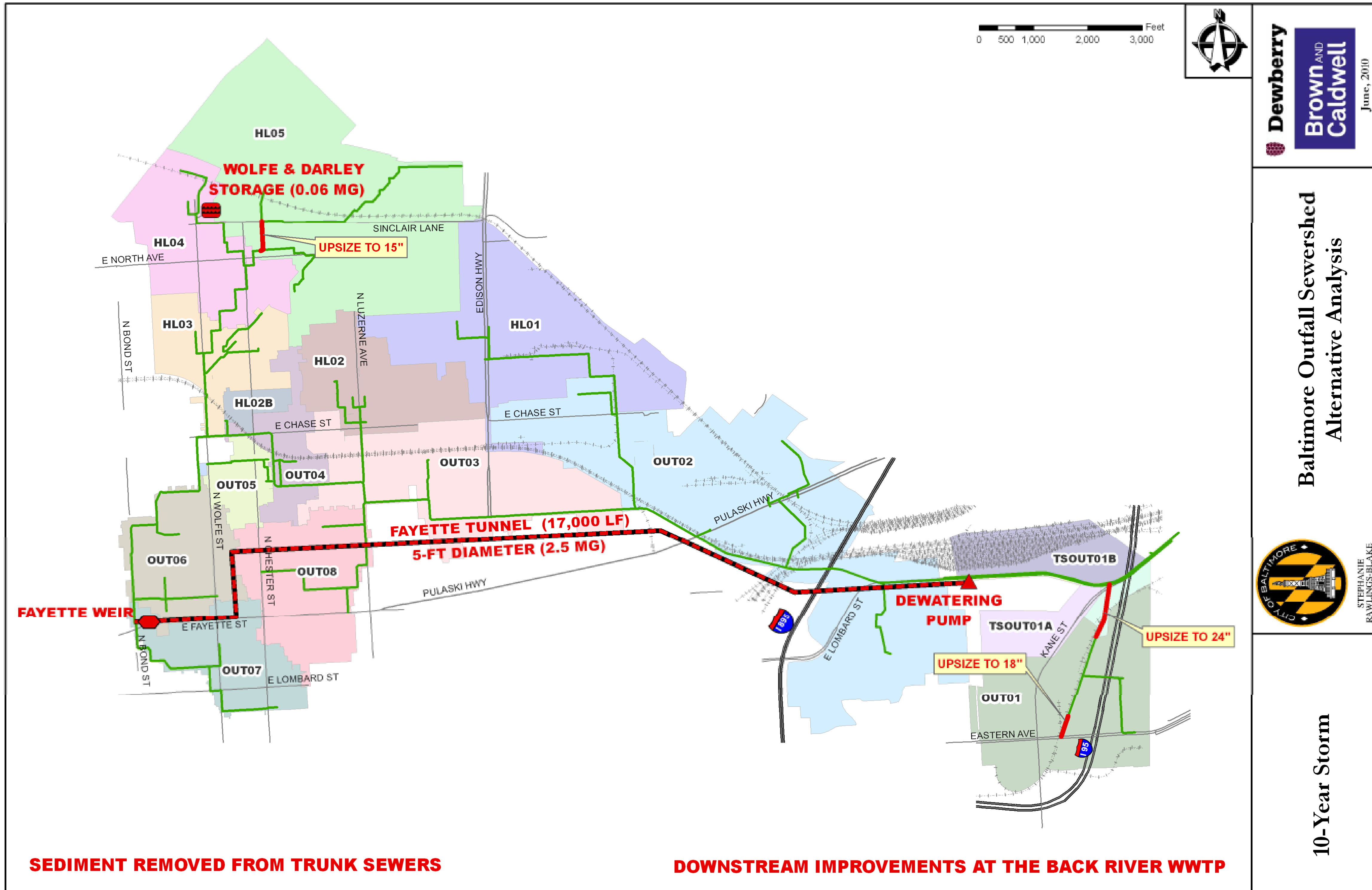
Additional branch sewer improvements are needed in meterbasin HL05 and OUT01. In the HL05 meterbasin, there is a simulated SSO along Sinclair Lane at Homestead Street (manhole S47MM_042MH) for the 10-year and larger events. This manhole is vulnerable to overflow because of a downstream hydraulic restriction along Collington Avenue. The size of the pipe along Collington Avenue needs to be increased from 12 to 15-inches to eliminate the SSO further upstream at Sinclair and Homestead. The 15-inch replacement pipe would run 592 LF along Collington Avenue from manhole S47MM_031MH (Sinclair & Collington) to manhole S45MM_025MH (in an alley west of Collington Avenue and north of North Avenue).

In the OUT01 meter basin, manhole S69G__005MH is the upstream end of the upper section in the model. This manhole, at Eastern Avenue, is the location of a small simulated overflow for the 10-year and larger events. The first pipe section in the model is a 15-inch pipe; all of the other pipe sections along this branch sewer are 18-inch diameter. The 10-year event requires a replacement pipe running approximately 400 LF from manhole S69G__005MH (at Eastern Avenue) to the next manhole north, S69G__008MH. The replacement pipe is upsized from 15 to 18-inches

Costs of the 10-year improvements are itemized in Table 5.4.4.

Based on the results of the sensitivity analysis for sub-optimal conditions in the AARR, the facilities needed for a 2-year level of protection in sub-optimal conditions are equivalent to those needed for the 10-year event with nominal conditions. Thus the major facilities costs presented in Table 5.4.4 are representative of the cost of facilities for a 2-year level of protection under sub-optimal conditions. These facilities are robust and provide protection with a greater degree of certainty. Even in extreme events greater than 10-year recurrence, these facilities are very effective in reducing the volume of SSOs, even if compete protection is not achieved.

Table 5.4.4 10-year Outfall Improvements Alternative 3: Tunnel, Sediment Removed						
Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.065	MG	$390,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
OUT01 Upper Section	18" Replacement Pipe	585	$/LF	400	LF	$234,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 5' x 17,000 LF	31.65	$/gal	2.5	MG	$79,023,110
	Dewatering Pump	2.84	$/gpd	2.5	MGD	$7,100,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$105,286,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$44,220,000
2008 Total Estimated Cost						**$149,506,000**
2009 Total Estimated Cost						**$159,971,000**
2010 Total Estimated Cost						**$171,169,000**
2011 Total Estimated Cost						**$183,151,000**
2012 Total Estimated Cost						**$195,972,000**
2013 Total Estimated Cost						**$209,690,000**
2014 Total Estimated Cost						**$224,368,000**
2015 Total Estimated Cost						**$240,074,000**
2016 Total Estimated Cost						**$256,879,000**
2017 Total Estimated Cost						**$274,861,000**



Map 5.4.2.3 10-year Improvements for Alternative 3

15-Year Improvements

For 15-year level of protection, a 6-foot tunnel (3.6 MG) is required at the Fayette relief point. The 15-year improvements are shown in Map 5.4.2.4.

Branch sewer facilities added for the 15-year level of protection include a second small storage tank and a replacement sewer in the HL04 meter basin. The storage tank is needed in the vicinity of North Avenue and Chester Street to reduce peak flows to the downstream sections of pipe. Not only do the larger events require additional storage at the Wolfe and Darley location, but 554 LF of pipe along Wolfe Street and Darley Street need to be upsized from 10 to 12 inches.

In meter basin HL05, the 12-inch sewer along Sinclair Lane needs to be upsized to 15-inches. This segment is 751 LF from manhole S47MM_042MH (Sinclair and Homestead) to Collington Avenue at manhole S47MM_031MH. In the upper section of meter basin OUT01, the 15-year event requires 1600 LF of pipe upsized to 21 inches from manhole S69G__005MH to manhole S69E__005MH. Costs of the 15-year improvements are itemized in Table 5.4.5

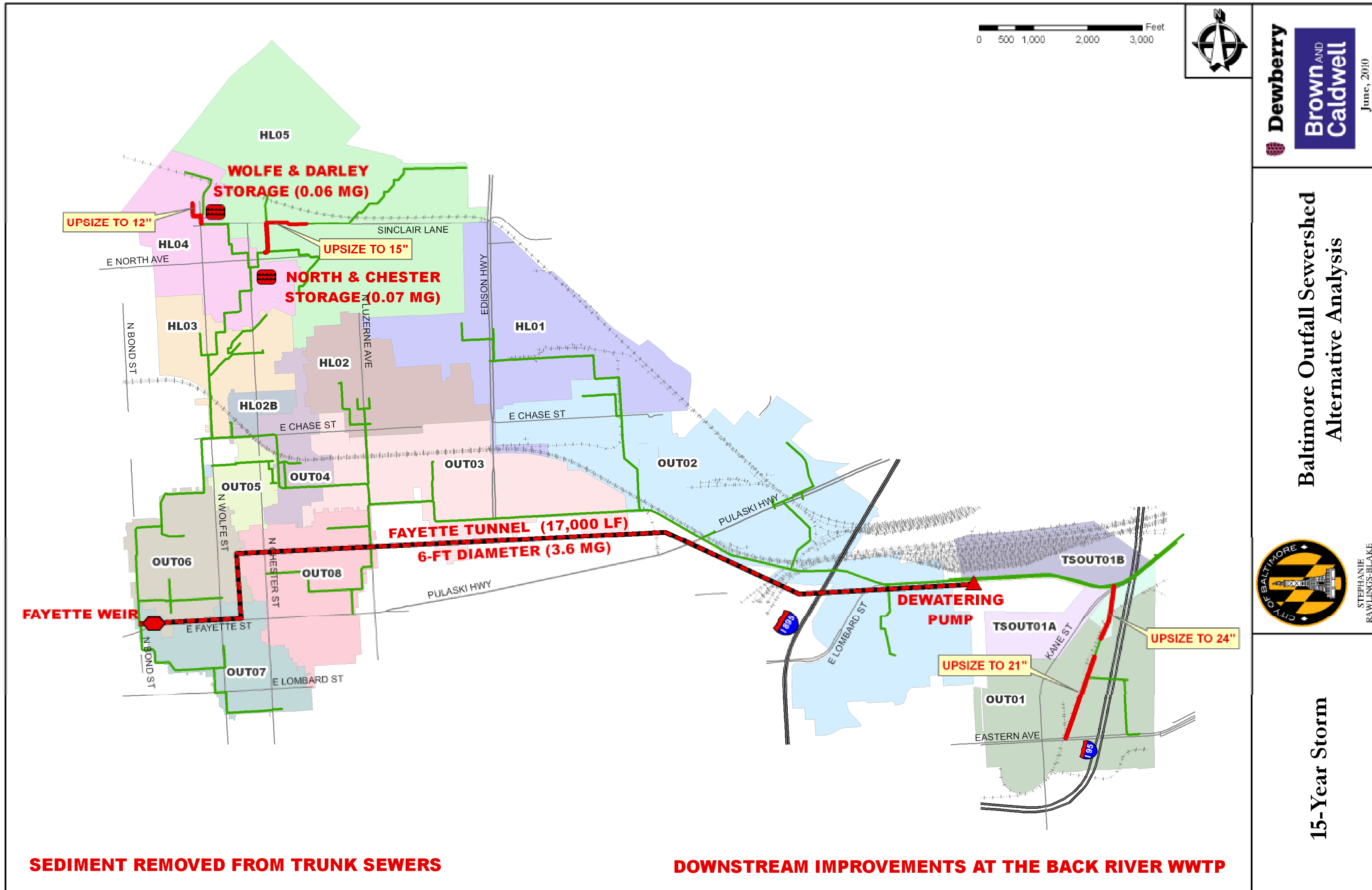
If a RDII reduction alternative were to be used instead of a storage tank, the peak flows from the Darley/Cliftview Avenue neighborhood would need to be reduced 30 to 50%. More extensive RDII reduction would be needed to provide the same benefit at the North and Chester storage tank.

The cost of RDII reduction was investigated. The Darley/Cliftview neighborhood has approximately 11,000 LF of sewers ranging in size from 8 to 24 inches. The cost to rehabilitate these sewers to reduce RDII would be approximately $3 million.

RDII reduction in the sewers upstream of the North/Chester overflow location would require rehabilitation of approximately 20,000 LF of pipe with a cost of $5.5 million. The total cost of RDII in the HL04 meter basin area would be approximately $8.5 million. The cost of RDII reduction is approximately an order of magnitude more than the cost of storage tanks, not considering the cost to convey and treat the extraneous RDII flow.

Table 5.4.5
15-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
			Branch Sewer Improvements			
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.058	MG	$348,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.073	MG	$438,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
			Major Relief Facilities			
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
			Sediment Cleaning in Trunk Sewers			
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$115,909,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,682,000
			2008 Total Estimated Cost			**$164,591,000**
			2009 Total Estimated Cost			**$176,112,000**
			2010 Total Estimated Cost			**$188,440,000**
			2011 Total Estimated Cost			**$201,631,000**
			2012 Total Estimated Cost			**$215,745,000**
			2013 Total Estimated Cost			**$230,847,000**
			2014 Total Estimated Cost			**$247,006,000**
			2015 Total Estimated Cost			**$264,296,000**
			2016 Total Estimated Cost			**$282,797,000**
			2017 Total Estimated Cost			**$302,593,000**



Map 5.4.2.4 15-year Improvements for Alternative 3

20-Year Improvements

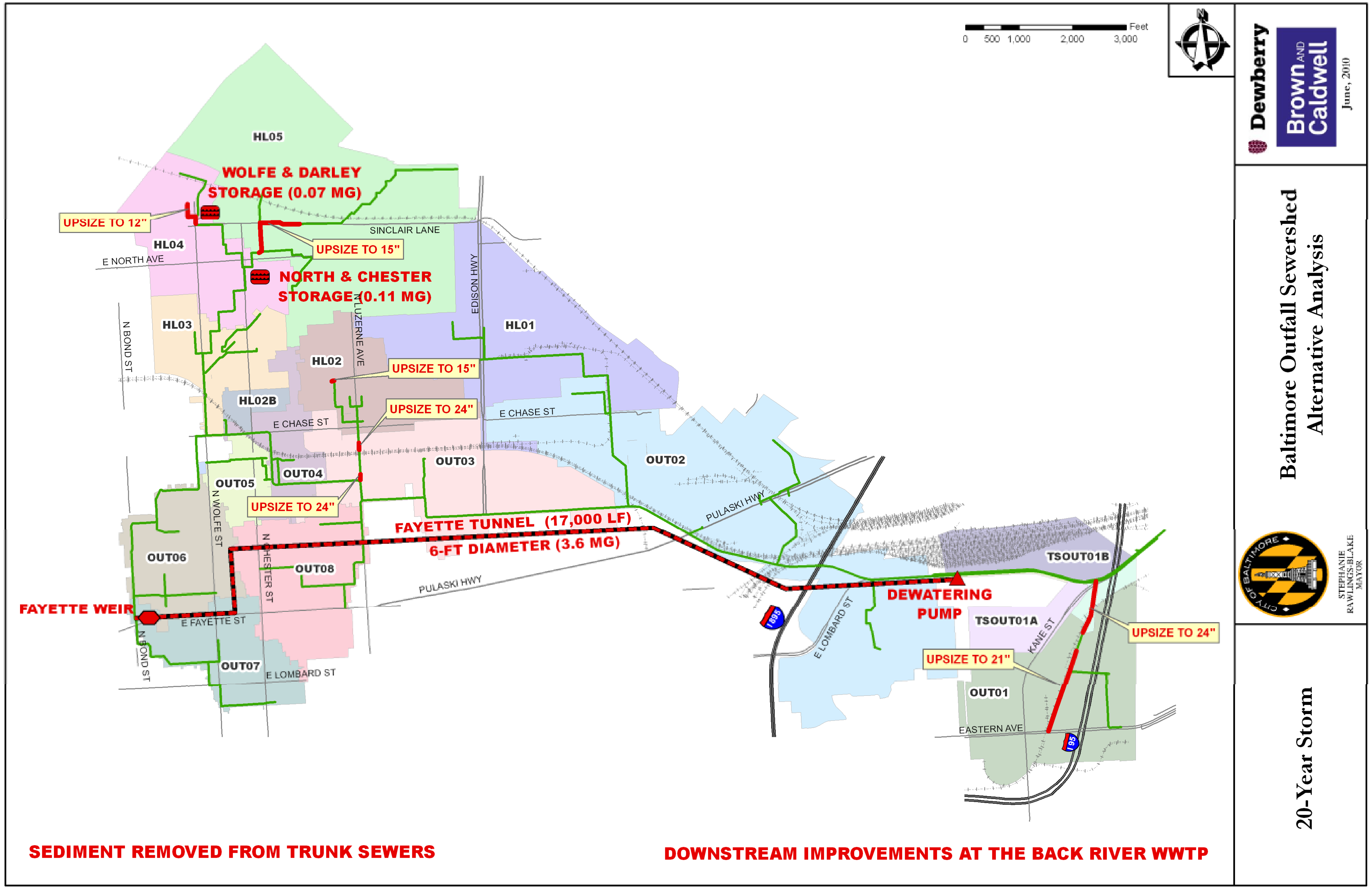
In general, the facilities needed for the 20-year event are very similar to those needed for the 15-year event. For 20-year level of protection, a 6-foot tunnel (3.6 MG) is required at the Fayette relief point. The 6-foot diameter tunnel, size for the 15-year level of protection, is also adequate to provide a 20-year level of protection. The 20-year improvements are shown in Map 5.4.2.5.

In the 20-year event, high peak flow rates cause surcharging all along the length of the HL02 branch sewer. To eliminate the SSO, upsizing the pipe near the downstream end of the branch sewer is recommended. The replacement pipes along Luzerne Street require upsizing from 15 inches to 18 inches. The first segment of the replacement runs 134 LF from manhole S49EE_004MH (Beryl Street) to manhole S49EE_021MH. The second segment of the replacement runs 137 LF from manhole S49CC_021MH to manhole S49CC_075UN (Ashland Street at the connection to the Outfall Interceptor). The total length of replacement along Luzerne Street is approximately 271 LF.

At the upstream end of the HL02 branch in the model there is a small overflow at Milton Street north of Preston Street (manhole S49GG_039MH) in the 20-year event. The short 10-inch sewer that crosses under the road needs to be upsized to 15 inches for 46 LF from manhole S49GG_039MH to manhole S49GG_027MH.

Costs of the 20-year improvements are itemized in Table 5.4.6.

Table 5.4.6 20-year Outfall Improvements Alternative 3: Tunnel, Sediment Removed							
Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL02 Milton Ave	15" Replacement Pipe	585	$/LF	46	LF	$26,910
HL02 Luzerne St	24" Replacement Pipe	1080	$/LF	271	LF	$292,680
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.074	MG	$444,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.107	MG	$642,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$116,529,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,942,000
2008 Total Estimated Cost						**$165,471,000**
2009 Total Estimated Cost						**$177,054,000**
2010 Total Estimated Cost						**$189,448,000**
2011 Total Estimated Cost						**$202,709,000**
2012 Total Estimated Cost						**$216,899,000**
2013 Total Estimated Cost						**$232,082,000**
2014 Total Estimated Cost						**$248,328,000**
2015 Total Estimated Cost						**$265,711,000**
2016 Total Estimated Cost						**$284,311,000**
2017 Total Estimated Cost						**$304,213,000**



Map 5.4.2.5 20-year Improvements for Alternative 3

Summary of Costs

Figure 5.4.2.6 shows the total costs for Alternatives 1, 2, and 3. Alternative 1 does not assume any downstream improvements at the Back River WWTP. This is the cost to manage the SSO problem within the Outfall sewershed with facilities located in the Outfall Sewershed alone. Alternative 1 does not address peak flows into the Back River WWTP that exceed the plant's existing treatment capacity.

Alternatives 2 and 3 assume that there are downstream improvements at the Back River WWTP, but the cost of those downstream improvements are not accounted for in this cost summary. The cost of Alternatives 2 and 3 are substantially lower than Alternative 1 because of the downstream improvements at the Back River WWTP. Even though the cost of Alternative 3 is greater than Alternative 2, the additional flexibility of the tunnel facilities merits consideration when choosing between the tank and tunnel concepts.

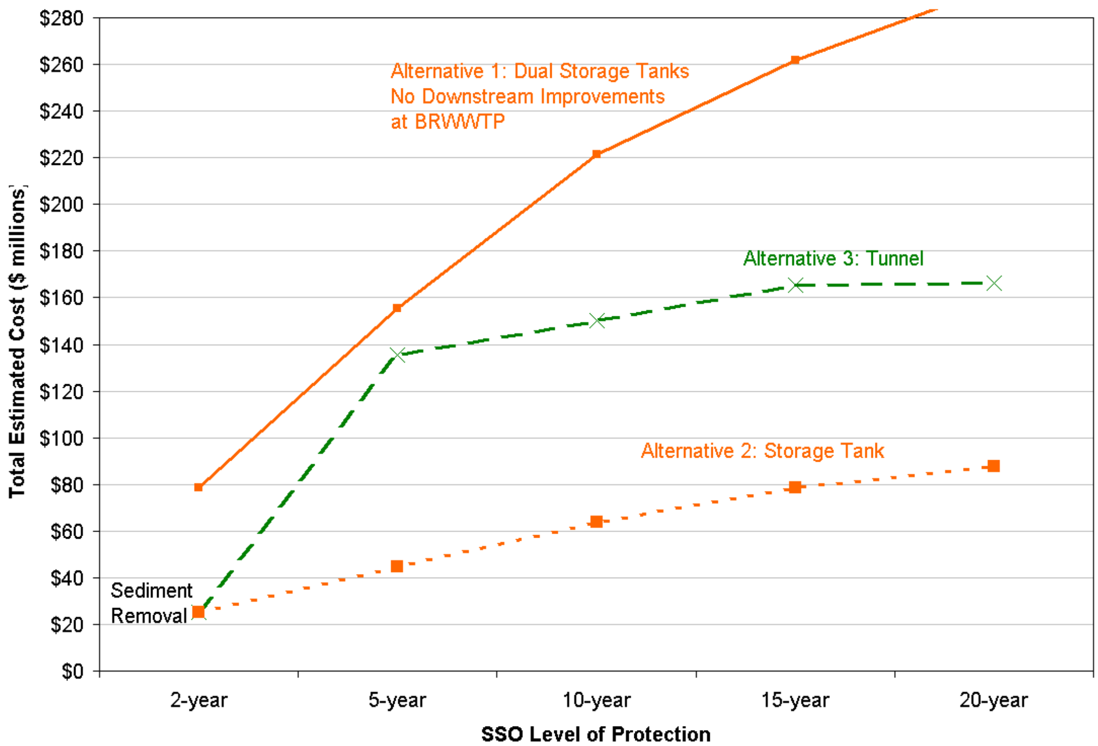


Figure 5.4.2.6 2008 Total Estimated Cost of Alternative 3

Construction costs were developed for all alternatives evaluated. To develop the estimated costs of construction, standard unit costs for sewer point repairs, sewer lining, sewer replacement, sewer cleaning, and manhole rehabilitation/replacement were provided by the City in 2008 dollars. The construction costs provided were fully loaded costs to address such items as mobilization, maintenance of traffic, paving restoration, bypass pumping and miscellaneous (non-sanitary) utility work. For costs not provided by the City (large diameter tunnels and pumping stations) recent projects within the

City and surrounding areas were reviewed to assist in estimating the most probable fully loaded cost of construction.

In addition to these construction costs, an additional 42 percent was added to accommodate engineering design, construction management/inspection, administration, post-award engineering services and contingencies. A 7 percent annual inflation rate is used to project costs for years beyond 2008.

Alternative 3 total estimated costs for the Outfall Sewershed improvements are summarized in Table 5.4.7 for the 2, 5, 10, 15, and 20-year events; the costs are inflated 7% per year for the recommended projects depending upon the year they might be implemented (from 2008 through 2017). The total estimated costs are under the column heading “Cumulative” in Table 5.4.7 for the 5, 10, 15, and 20-year events. The “Additional” cost column in the table is the incremental cost of facilities from one design storm level of protection to the next.

Table 5.4.8 is a summary of total estimated cost normalized by the volume of SSO removed. The units are dollars per gallon of SSO removed. The cumulative cost divided by the cumulative SSO volume removed is a direct normalization of the total cost by the total SSO volume. For example: The 2-year facilities removed 29.3 MG of SSO at a cost of $24 million; thus the unit cost is $0.83 per gallon of SSO removed. The 2-year facilities eliminate all of the SSOs in the 2-year event.

Incremental normalized cost values are also given in the table under the “Additional” columns. The additional costs per additional gallon of SSO volume removed were developed in the following manner: The 2-year facilities are effective in removing much of the SSO volume for the 5-year event, but the remaining SSO volume is 0.32 MG with the 2-year facilities in place. The additional cost of the 5-year facilities is $111 million compared to the 2-year facilities. The 5-year facilities are needed to remove the 0.32 MG of SSO that would remain if the 2-year facilities were in place. Therefore, the normalized additional cost is $346 per gallon of additional SSO removed.

The step wise progression was used to determine the additional SSO that could be removed by the 10-year facilities compared to the SSO remaining with the 5-year facilities. The normalized additional cost is $730 per gallon of additional SSO removed to reach the 10-year level of protection.

Likewise, the analysis determined the additional costs and the additional SSO volumes removed by the 15 and 20-year facilities. The additional volumes removed in these cases are negligible; therefore, the normalized additional costs are undefined.

The additional SSO removed is a relatively small volume because facilities sized for a smaller event are very effective at removing most of the SSO volume in a larger event, even though they may not be adequate to remove 100% of the SSO volume. As a result, the normalized costs ($/gallon) to remove the additional SSO volumes are extremely high.

Table 5.4.7
Total Estimated Outfall Improvement Costs

Projected Year	2-yr Cost	5-yr		10-yr		15-yr		20-yr	
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$24,282,000	$110,629,000	$134,911,000	$14,595,000	$149,506,000	$15,085,000	$164,591,000	$880,000	$165,471,000
2009	$25,982,000	$118,373,000	$144,355,000	$15,616,000	$159,971,000	$16,141,000	$176,112,000	$942,000	$177,054,000
2010	$27,801,000	$126,659,000	$154,460,000	$16,709,000	$171,169,000	$17,271,000	$188,440,000	$1,008,000	$189,448,000
2011	$29,747,000	$135,525,000	$165,272,000	$17,879,000	$183,151,000	$18,480,000	$201,631,000	$1,078,000	$202,709,000
2012	$31,829,000	$145,012,000	$176,841,000	$19,131,000	$195,972,000	$19,773,000	$215,745,000	$1,154,000	$216,899,000
2013	$34,057,000	$155,163,000	$189,220,000	$20,470,000	$209,690,000	$21,157,000	$230,847,000	$1,235,000	$232,082,000
2014	$36,441,000	$166,024,000	$202,465,000	$21,903,000	$224,368,000	$22,638,000	$247,006,000	$1,322,000	$248,328,000
2015	$38,992,000	$177,646,000	$216,638,000	$23,436,000	$240,074,000	$24,222,000	$264,296,000	$1,415,000	$265,711,000
2016	$41,721,000	$190,082,000	$231,803,000	$25,076,000	$256,879,000	$25,918,000	$282,797,000	$1,514,000	$284,311,000
2017	$44,641,000	$203,388,000	$248,029,000	$26,832,000	$274,861,000	$27,732,000	$302,593,000	$1,620,000	$304,213,000

Table 5.4.8
Total Estimated Outfall Improvement Costs per Gallon SSO Removed

SSO Volume (MG)	**Upstream Improvements 2-yr**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Remaining with 2-yr Facilities	Upstream Improvements	Remaining with 5-yr Facilities	Upstream Improvements	Remaining with 10-yr Facilities	Upstream Improvements	Remaining with 15-yr Facilities	Upstream Improvements
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**
SSO Volume Removed (MG)	**2-yr**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Additional SSO Removed by 5-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 10-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 15-yr Facilities	Cumulative SSO Removed	Additional SSO Removed by 20-yr Facilities	Cumulative SSO Removed
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**
Projected Year	**2-yr Cost**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$0.83	$346.00	$2.98	$730.00	$2.62	undefined	$2.59	undefined	$2.44
2009	$0.89	$370.00	$3.19	$781.00	$2.80	undefined	$2.77	undefined	$2.61
2010	$0.95	$396.00	$3.41	$835.00	$3.00	undefined	$2.96	undefined	$2.79
2011	$1.02	$424.00	$3.65	$894.00	$3.21	undefined	$3.17	undefined	$2.99
2012	$1.09	$453.00	$3.90	$957.00	$3.43	undefined	$3.39	undefined	$3.19
2013	$1.16	$485.00	$4.18	$1,024.00	$3.67	undefined	$3.63	undefined	$3.42
2014	$1.24	$519.00	$4.47	$1,095.00	$3.93	undefined	$3.88	undefined	$3.66
2015	$1.33	$555.00	$4.78	$1,172.00	$4.20	undefined	$4.16	undefined	$3.91
2016	$1.42	$594.00	$5.12	$1,254.00	$4.50	undefined	$4.45	undefined	$4.19
2017	$1.52	$636.00	$5.48	$1,342.00	$4.81	undefined	$4.76	undefined	$4.48

References

(1) National Weather Service, 2004. "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 3, G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
Depth, duration, and frequency data for Baltimore WSO City available online from the National Weather Service, Hydrometeorological Design Studies Center, Precipitation Frequency Data Server
http://dipper.nws.noaa.gov/hdsc/pfds/orb/md_pfds.html

(2) U.S. Soil Conservation Service. Technical Release 55: Urban Hydrology for Small Watersheds. USDA (U.S. Department of Agriculture). June 1986. Available from NTIS (National Technical Information Service), NTIS # PB87101580. Also available on the web in .pdf format at
http://www.info.usda.gov/CED/ftp/CED/tr55.pdf

6.0 Geographic Information System (GIS)

6.1 Overview of GIS

The City of Baltimore maintains a robust Geographic Information System (GIS) representing the wastewater infrastructure. The GIS is housed in an ESRI format Geodatabase and leverages the enterprise capabilities of ArcSDE. At the time of this report, this data was compiled using ArcGIS version 9.2. An integral part of the sewershed study is the update of the GIS to represent the existing conditions at the time of the study. These updates provided to the City were considered "Core" data deliveries as they are the primary or core repository of data representing the wastewater infrastructure. This is in comparison to "non-core" data which was the supplemental data provided to the City such as manhole inspection reports, CCTV video, etc. This section describes the City's GIS system; describes the methods and procedures used during the project to update the system; and the quality assurance procedures performed to verify the accuracy of the work performed.

The wastewater utility geodatabase is comprised of three thematic groups of features:

- Lines Thematic Group – contains wastewater features that can be represented as lines whose direction indicates the direction of flow. These line features make up the foundation of the wastewater network. All features in this thematic group participate in the geometric network. These features include:

    - House Connection (line)
    - Sewer (line)

- Features Thematic Group – contains wastewater features that can be represented as points, lines and/or polygons. The features in this thematic group do not affect flow and will not participate in the geometric network. Traces and other network analysis operations do not consider these entities, yet they are captured in the database to provide a more complete representation of the system. These features include:

    - Casing (polygon)
    - Facility (polygon)
    - Lamphole (point)
    - Manhole Cover (point)
    - Structure (polygon)

- Devices Thematic Group – contains wastewater features that can be represented as points. All features in this thematic group participate in the geometric network. These features include:

    - Manhole Junction (point)

- Meter Station (point)
- Pump Station (point)
- Treatment Plant (point)
- Bend (point)
- Valve (point)
- House End (point)
- House Intersection (point)
- House Sewer Intersection (point)
- Sewer End (point)
- Sewer Intersection (point)

The Outfall Sewershed consisted almost entirely of gravity systems, and therefore contained no pressure systems or related features (such as bends, pump stations, etc.).

The following graphic summarizes the feature objects in the City's wastewater GIS.

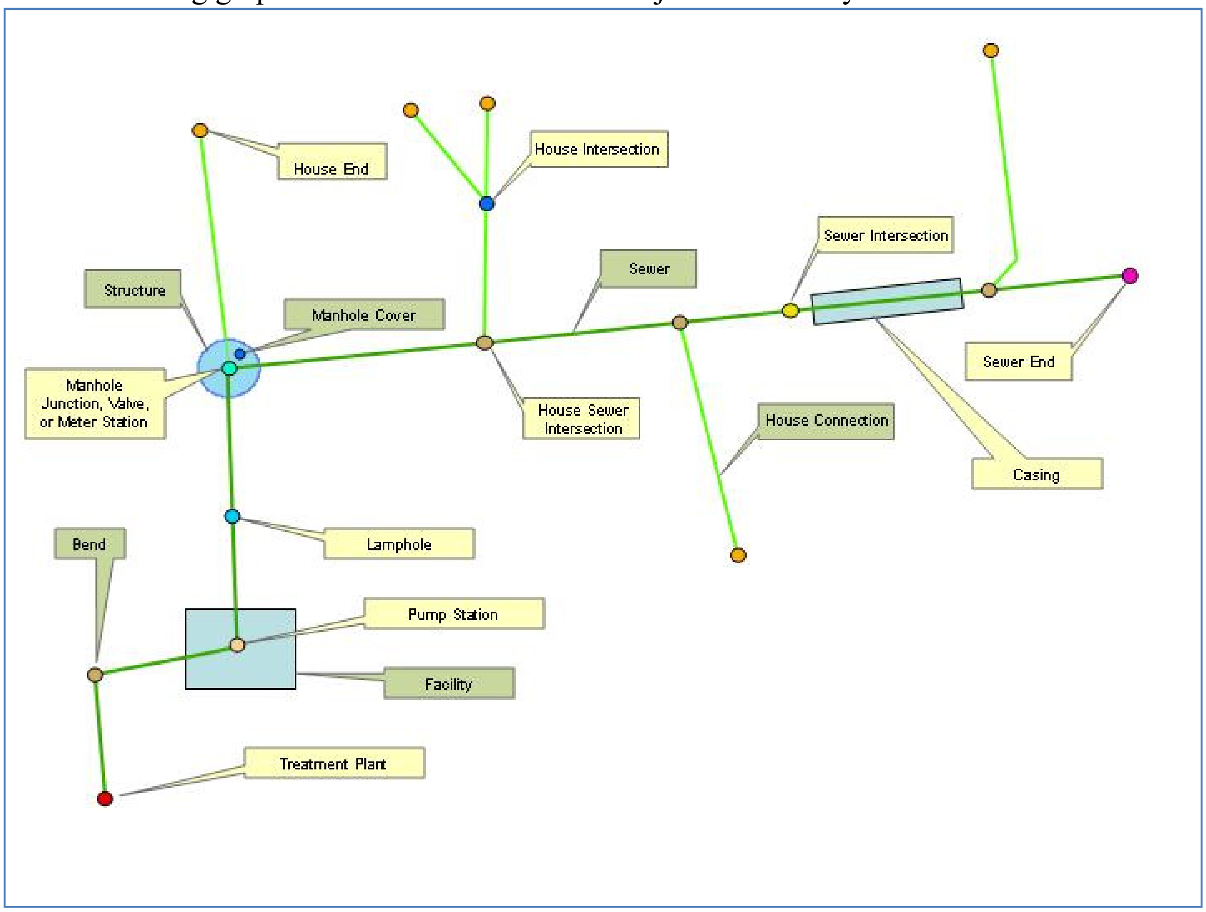


Figure 6.1 - Feature Objects in the City's Wastewater GIS

6.2 Field Data and GIS Integration

The Sewershed Study and Evaluation project involved extensive field activities which generated significant amounts of non-core data to be used to update the core GIS. Specifically, the non-core data generated was:

- Manhole Inspection Data
- GPS Survey Data
- Closed Circuit Television (CCTV) Inspection Data
- Smoke Testing
- Dyed Water Testing Data

The majority of the spatial and attribute edits made to the wastewater geodatabase were based on information extracted from these non-core datasets, namely the manhole inspections, and GPS survey data. When current conditions could not be established through these sources, additional engineering contract document research was performed to populate the GIS. The following is further description regarding the field collected data and its use in updating the GIS.

Manhole Inspections

Manhole inspections were performed on 1845 manholes in the Outfall sewershed. Information was collected using a custom designed Manhole Inspection Application Software (MIAS) application. MIAS allows field crews to collect detailed attribute information about the physical characteristics of a manhole, its sewer connections, and the manhole's surrounding environment. In addition to characteristics such as size, shape, and material, the application records the condition and infiltration properties of the manhole's features. The MIAS application captures inventory and condition information for the following manhole components:

- Location
- Environment
- Cover
- Frame
- Chimney/Stack
- Corbel
- Barrel
- Bench
- Channel
- Pipe Connections

The unique identifier used in both the GIS and MIAS datasets is the MANHOLE_ID field. This common field allowed for database joins which facilitated integration of the manhole inspection field information directly into wastewater feature attribute fields.

Roughly 11,707 manhole inspection photos were taken during the manhole inspections in the Outfall sewershed. The MIAS application and other GIS tools provided easy access to these photos for use in checking and validating the manhole information being entered into the database.

GPS Manhole Surveys

A total of 1,811 survey-grade GPS survey locations of manhole covers were completed during the project, representing 93% of all City-owned manholes. The remaining manholes were not surveyed due to access issues. These GPS locations were used to position key manhole features and to establish the rim elevation stored in the manhole cover GIS feature class.

The GPS rim elevations were used along with depths measured during the manhole inspection, from the rim down to the invert of each pipe connecting manhole, to establish pipe invert elevations in the Sewer feature layer.

Rim elevations for manholes that were not GPS surveyed were extracted from construction drawings where available. If rim elevations were not available on the contract drawing, the raw invert elevations from the construction drawings were used, with those elevations being converted from the City's vertical datum to NAVD88 using the provided conversion factor.

CCTV Inspections

The Outfall sewershed study plan team completed roughly 2,107 individual CCTV sewer inspections. The up and down nodes for each CCTV survey were verified that they link to a valid GIS manhole, lamphole or SewerEnd features that represent the starting and ending locations of the survey.

With the data relationship established, the CCTV surveys, manhole inspections (MIAS database) and the GIS were compared to assist in GIS attribute updating.
The CCTV surveys were invaluable in the GIS updating process by enabling Engineers and GIS technicians to:

- Locate previously unknown buried manholes and to incorporate them into the GIS at their proper location.
- Establish the existence of manholes in the GIS
- Identify the proper location of changes or fixtures in the system:
    - Size changes
    - Material changes
    - Angular changes
    - Tees and Wyes (sewer mains connecting without a manhole)

Inspectors also recorded changes between actual field conditions and the current GIS information on paper plots of the GIS data. The main value of this was the ability for the CCTV inspectors to validate or correct pipe connectivity throughout the Sewershed. Using these marked-up plats provided a convenient medium to record additional remarks that were then later modified in the GIS by technicians.

Smoke and dyed water testing

Smoke and dyed water testing were performed in areas where the cross-connections with storm drains were suspected and continuity of the pipe network could not be determined through other methods. Reports including photo documentation were prepared and were then used by technicians to appropriately modify the GIS data. In total, 119 smoke testing reports were generated and 24 dyed water testing reports were generated for the Outfall Sewershed.

6.3 Office Research and GIS Updates

The compilation of field collected data allowed GIS technicians to update a significant amount of the GIS representation of the wastewater infrastructure. Prioritization of the applicability of the variety of sources was performed on an attribute by attribute basis based upon the guidance provided by the City's Baltimore Sewer Evaluation Standards manual (BaSES). Some features or attributes could not be adequately quantified using the collected field information and required additional research of Baltimore's record plat maps and engineering contract drawings.

Using standard ESRI editing functionality in the ArcGIS platform as well as custom tools for GIS updates, GIS technicians utilized the sources available to them to update the wastewater geodatabase. As tiles in the City's standard grid index were completed and quality assurance approved, the data was synchronized back to the City for quality control review by the data clearinghouse.

6.4 QA/QC Review and Procedures

A variety of procedures were performed for quality assurance and quality control of the wastewater geodatabase.

- Oversight and manual spot checks by engineers and GIS analysts were performed for quality assurance.
- ArcInfo topology checks to verify feature topology; feature snapping; flow tracing; and location of duplicate features.
- Database queries to compare the GIS datasets with the other non-core data sources were executed to review for anomalies.
- An automated suite of 147 quality control tests built in the ESRI Production Line Tool Set (PLTS) platform were run against the dataset both by the sewershed consultant as well as the data clearinghouse. These tests perform a variety of checks on features and feature attributes, including: domain validation, attribute, logical, spatial, and topologic.

6.5 GIS Certification

The Outfall Sewershed team has followed the processes described above and those described in more detail in the City BaSES manual to update the City of Baltimore's wastewater GIS for the Outfall Sewershed. The City of Baltimore and the Outfall Sewershed team are hereby certifying that the GIS data represented in the Outfall sewershed portion of the City's GIS provides the necessary data for the adherence of Paragraph 14 Information Management System Program.

The Outfall Sewershed portion of the City's GIS is the best assessment of current conditions achievable with the available technology and source data. Current conditions are defined as of 01/25/2010. Furthermore, the City of Baltimore has instituted processes to ensure that should changes to the sewer infrastructure in the Outfall Sewershed occur, the GIS will be updated within 90 days of the changes.